<u>FIRST AMENDMENT TO</u>
<u>DISPOSITION AND DEVELOPMENT AGREEMENT</u>

This FIRST AMENDMENT TO DISPOSITION AND DEVELOPMENT AGREEMENT ("First Amendment") is dated January ____, 2021, and is entered into by and among the FAIRFIELD HOUSING AUTHORITY, a public entity (the "FHA"), and AFFORDABLE HOUSING ALLIANCE II, INC., a Colorado nonprofit corporation, ("Integrity") and SUTTON PLACE DEVELOPMENT CORP, a California corporation ("Sutton"). Integrity and Sutton are herein collectively, and jointly and severally referred to as, the "Developer". FHA and Developer are sometimes individually referred to herein as "Party" and collectively as "Parties."

RECITALS

A. The Parties entered into a Disposition and Development Agreement dated February 10, 2020 ("DDA"), in which the FHA agreed to sell certain real property ("Property") to Developer upon satisfaction of certain conditions, and which requires Developer to timely develop a project on the Property. Capitalized terms used but not defined herein shall have the meaning set forth in the DDA.

B. Developer has investigated the condition of the Property, and has approved such condition, except that Developer has discovered underground concrete storage tanks, which Developer will need to mitigate to enable the project to be developed. Developer has obtained a geotechnical report from GEOCON dated May 7, 2020, a copy of which is attached hereto as <u>Exhibit "A"</u>; the work that Developer must complete ("Work") is described in the report attached hereto as <u>Exhibit "A"</u>.

C. The Purchase Price in the DDA was intended to be the fair market value of the Property and was determined by appraisal, but the appraiser did not consider the costs of the Work; consequently, the Parties agree that the fair market value of the Property is the Purchase Price in the DDA less the costs of completing the Work.

D. The Parties desire to provide for Escrow Holder to hold in escrow from the purchase price funds, after the closing of the conveyance of the Property under the DDA, the sum of $400,000 (the "Soils Engineering Funds"), and make disbursements thereof to pay for costs of the Work until the date that is 180 days after the Close of Escrow under the DDA, as extended by Force Majeure delays (the "Work Completion Deadline"), at which point any remaining Soils Engineering Funds shall be disbursed to the FHA as the remainder of the purchase price under the DDA.

E. FHA and Developer also desire to amend the DDA in order to: (i) replace the Schedule of Performance (Exhibit "C" to the DDA) with the revised Schedule of Performance attached hereto and captioned <u>Exhibit "C"</u>; (ii) replace the Scope of Development (Exhibit "D" to the DDA) with the revised Scope of Development attached hereto and captioned <u>Exhibit "D"</u>; (iii) add a Project Budget (attached hereto and captioned <u>Exhibit "F"</u>) as Exhibit "F" to the DDA (which satisfies the budget requirement in Section 2.4.1 of the DDA); (iv) extend the deadline for Close of Escrow to February 10, 2022; (v) provide that the FHA shall have a right to review and reasonably approve the plans and specifications for the Project; and (vi) provide that the Developer will apply for 4% low income housing tax credits (unless alternative affordable housing program financing has been secured) in each of the next four application rounds until such tax credits are awarded for the Project, and provide that such award of tax credits is a condition to the closing.

NOW, THEREFORE, in consideration of the foregoing recitals, the mutual agreements/amendments herein, and other consideration, the sufficiency of which is hereby acknowledged, the Parties hereby agree as follows.

AGREEMENT/AMENDMENTS

1. Supplemental Escrow Instructions. Upon the execution of this First Amendment, the parties shall deliver a copy of this executed First Amendment to Escrow Holder, and this First Amendment shall then constitute supplemental escrow instructions of the Parties to Escrow Holder, but the Parties shall execute and deliver to Escrow Holder the form of "Instructions to Withhold Funds After Close of Escrow (Repairs)" attached hereto as Exhibit "B" and such additional reasonable escrow instructions required by Escrow Holder provided they do not conflict with this First Amendment.

2. Construction Contract. Developer shall select one contractor to be its primary contractor for the Work and its bid, and Developer shall engage that contractor; however, FHA acknowledges that other consultants, engineers and contractors may be engaged to perform certain aspects of the Work. Developer shall deliver a copy of all executed contracts necessary for the Work to FHA prior to and as a condition to the close of escrow. Nothing herein shall delay the deadline for Close of Escrow, or FHA's rights to terminate the DDA if the Close of Escrow does not timely occur.

3. Plans and Specifications. Whenever plans and specifications or revisions thereof are delivered to the City of Fairfield, a separate copy shall be delivered to the FHA, and FHA shall have the right to review and reasonably approve the plans and specifications, it being acknowledged that FHA's interest in the plans and specifications as a housing authority differs from the City of Fairfield's interest, rights and obligations as to the plans and specifications as a governmental "permitting/land use" entity. FHA's review and approval thereof shall be a condition to the Close of Escrow.

4. Withholding of Funds. At the Close of Escrow under the DDA, Escrow Holder shall withhold, from the funds received from Developer to pay the purchase price under the DDA, a sum equal to FOUR HUNDRED THOUSAND AND NO/100 DOLLARS ($400,000.00) (the "Soils Engineering Funds"), and shall disburse such sum as set forth in the "Instructions to Withhold Funds After Close of Escrow (Repairs)" described in Section 1 above.

5. Release/Disbursements of the Soils Engineering Amount. Until the Work Completion Deadline, FHA and Developer shall instruct Escrow Holder in writing to disburse to Developer from time to time (but not more often than once every 30 days) portions of the Soils Engineering Funds from Escrow for costs of the Work, subject to Section 6 below.

6. FHA/Developer Process for Disbursements. Disbursements of Soils Engineering Funds shall be made to Developer per the "Instructions to Withhold Funds After Close of Escrow (Repairs)" attached hereto as Exhibit "B". FHA shall have the right to approve in good faith all disbursements of the Soils Engineering Funds, but shall not be obligated to approve disbursements more than once every thirty (30) days. In order to obtain a disbursement, Developer must provide

to FHA in writing the amount of the requested disbursement together with reasonable evidence of the costs of the Work that have been performed which are to be paid or reimbursed by the disbursement. FHA reserves the right to do a field inspection of the completed work being submitted for disbursement.

7. Work Completion Deadline. Developer shall complete (or cause the contractor to complete) the Work on or before the Work Completion Deadline, and shall provide reasonable evidence to FHA of any Force Majeure delays. FHA shall then confirm any Force Majeure extensions and the Work Completion Deadline in writing to Escrow.

8. Release of Unused Soils Engineering Funds to FHA. Upon the earlier of the Work Completion Deadline or the date on which the Work shall have been completed, any remaining Soils Engineering Funds shall be disbursed to the FHA as the remainder of the Purchase Price upon written demand of FHA to Escrow Holder (which shall include a statement that the Work has been completed or that the Work Completion Deadline has passed, as applicable).

9. Costs of Extended Escrow. Developer and FHA shall each pay fifty percent (50%) of the costs and charges of Escrow Holder relating to the escrowed funds and disbursements thereof. If FHA engages a consultant/contractor to review disbursement requests, FHA shall pay the costs of such consultant/contractor.

10. Extension of Closing Deadline. Section 2.2 of the DDA is hereby amended by deleting the sentence: "Escrow shall close (the "Close of Escrow") on or before the date that is one (1) calendar year after the Effective Date." and replacing it with "Escrow shall close (the "Close of Escrow") on or before February 10, 2022."

11. Applications for Affordable Housing Financing. The following is added to Section 2.4 (FHA Conditions to Close of Escrow):

"2.4.8 Developer shall have applied for 4% tax credits at every opportunity after the date of the First Amendment of DDA or alternative affordable housing financing program, and shall have provided FHA with reasonable evidence thereof."

The following is added to Section 2.5 (Developer Conditions to Close of Escrow):

"2.5.7 Developer shall have been awarded 4% tax credits or secured financing through an alternative affordable housing financing program for the Project."

12. Purchase Price. The parties acknowledge that they have agreed on a Purchase Price of $1,500,000 based on an appraisal dated September 12, 2020 as the fair market value of the Property, but that if the transaction does not close by March 12, 2021, the second paragraph of Section 2.1.2 of the DDA shall apply, it being the intent of FHA that the Property not be sold for less than its appraised fair market value as of the date of closing, and it being the intent of both FHA and Developer that prevailing wages not apply to the costs of constructing the Project.

13. Time of Essence. Time is of the essence of every provision hereof in which time is a factor.

14. Effect on Agreement. All terms and conditions of the Agreement that are not modified by this First Amendment shall remain unmodified, in full force and effect and binding on the Parties.

15. Conflict. In the event of a conflict between the terms and conditions of this First Amendment and the terms and conditions of the Agreement, the terms and conditions of this First Amendment shall control.

16. Counterparts. This First Amendment may be signed in counterparts (including facsimile or electronic counterparts), each of which shall be deemed an original, and all such counterparts, when taken together, shall constitute one agreement.

17. Governing Law. The laws of the State of California shall govern the interpretation and enforcement of this First Amendment, without application of conflicts or choice of laws principles.

18. Interpretation. The terms, provisions, conditions, covenants, restrictions and agreements contained in this First Amendment shall not be construed in favor of or against any Party, but shall be construed as if each Party prepared this First Amendment.

19. Entire Agreement. The Agreement, as amended by this First Amendment, represents the entire understanding between the Parties as to the subject matter of the Agreement, as so amended.

The Parties have signed and entered into this First Amendment as of the date first set forth above.

DEVELOPER:

AFFORDABLE HOUSING ALLIANCE II, INC.

By: ______________________
Philip Wood
President

SUTTON PLACE DEVELOPMENT CORP

By: ______________________
Patrick Morrill
President

FHA:

FAIRFIELD HOUSING AUTHORITY

By: ______________________

____
Executive Director

APPROVED AS TO FORM:

____

____

EXHIBIT "A"

GEOCON REPORT, WITH DESCRIPTION OF WORK

(Attached.)

GEOCON
CONSULTANTS, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. E9186-04-01
June 30, 2020

Integrity Housing
4 Venture, Suite 295
Irvine, California 92618

Attention: Mr. Phil Wood

Subject: 5-ACRE MULTI-FAMILY RESIDENTIAL DEVELOPMENT
NORTHWEST CORNER OF WOOLNER AVENUE AND GREGORY LANE
FAIRFIELD, CALIFORNIA
GEOTECHNICAL INVESTIGATION

Dear Mr. Wood:

In accordance with your authorization, we have performed a geotechnical investigation for the subject multi-family residential project planned in Fairfield, California. Our investigation was performed to observe the soil and geologic conditions that may impact site development for the project as presently planned. The accompanying report presents the results of our investigation and conclusions and recommendations pertaining to the geotechnical aspects of the proposed project. The findings of this study indicate the site is suitable for development as planned provided the recommendations of this report are implemented during design and construction.

If you have any questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Sincerely,

GEOCON CONSULTANTS, INC.

DRAFT

Andre E. Ashour, PE
Senior Project Engineer

DRAFT

Shane Rodacker, GE
Senior Engineer

(1/e-mail) Addressee

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for a planned multi-family residential development in Fairfield, California. The purpose of this investigation was to evaluate the subsurface soil and geologic conditions in the area of planned development and provide conclusions and recommendations pertaining to the geotechnical aspects of project design and construction, based on the conditions encountered during our study.

The scope of this investigation included field exploration, laboratory testing, engineering analysis and the preparation of this report. Our field exploration consisted of 13 test pits performed on March 4 and 5, 2020 to depths ranging from approximately 5½ to 16 feet below the existing surface grade, 5 soil borings drilled on April 20, 2020 to depths ranging from approximately 5 to 40 feet, and 6 Cone Penetrometer Tests (CPTs) advanced on March 17, 2020 to a depth of about 50 feet. The locations of our explorations are depicted on the Site Plan, Figure 2. A detailed discussion of our field investigation, boring and test pit logs and CPT profiles are presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to evaluate pertinent geotechnical parameters. In addition, four soil samples were submitted to our laboratory for screening-level corrosion testing. Appendix B presents the laboratory test results in tabular format and graphical format. Soil boring logs from a previous study by others are included in Appendix C. Appendix D presents output from our liquefaction analysis.

The opinions expressed herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the *List of References* section.

If project details vary significantly from those described herein, Geocon should be contacted to determine the necessity for review and possible revision of this report.

2. SITE BACKGROUND AND PROJECT DESCRIPTION

The site is an approximately 5-acre parcel (Solano Co. APN 0031-201-030) at the northwest corner of Woolner Avenue and Gregory Lane in Fairfield (see attached Vicinity Map, Figure 1). Development in the immediate site vicinity includes single family residential to the east and south and an apartment community immediately to the north. A parking lot for Allan Witt Park borders the western edge of the site.

A City of Fairfield water treatment plant operated at the site from the 1960s into the 1990s. The site has more recently been used by the City as a materials storage yard. The former water treatment facility included a pumping plant, clarifier, two large water storage reservoirs in the east-central portion of the site, and two sludge ponds along the western margin. The storage reservoirs were approximately 16 feet deep and were backfilled sometime in the 1990s. Documentation relative to demolition of the water plant and backfill of the reservoirs and sludge ponds is not available. Based on our subsurface exploration, the concrete reservoir sidewalls and floors were buried intact.

Web-based mapping indicates the ground surface at the site is relatively flat with existing grades of approximately 18 to 20 feet MSL. We observed scattered end-dumped stockpiles throughout the site. We did not investigate the constituents of those stockpiles.

Based on the information provided by Sutton Place Development Corporation, several site layouts are under consideration. In general, the project will redevelop the site with multi-family residential buildings up to three stories in height at the western, eastern and southern margins of the site. The interior of the site will receive amenity buildings, a community pool and possibly additional multi-family buildings. All buildings will be generally constructed at-grade with no subterranean levels and we have assumed the structures will be wood-framed. At-grade parking and driveways are planned throughout the site. We anticipate the project will also include new underground utilities and surficial site improvements e.g. landscaping and exterior flatwork. A Site Plan is presented as Figure 2 and reflects one of the potential site development alternatives.

Structural loads are not currently known for the proposed structure(s); however, structural loads are expected to be typical of similar type structures. Notwithstanding remedial excavations for geotechnical purposes (removing the existing undocumented fill) or excavations for the community pool, we have assumed that project grading will consist of cuts and fills of approximately two feet to attain design subgrade elevation for the new building pads.

3. GEOLOGIC SETTING

Fairfield is located at the western margin of the Great Valley Geomorphic Province of California, more commonly known as the Central Valley. The valley is a broad lowland between the Sierra Nevada to the east and Coast Ranges to the west. The Central Valley has been filled by a sequence of deep alluvial deposition derived from weathering processes in surrounding mountain ranges and foothills. The weathering and subsequent deposition within the valley has resulted in alluvial deposits that can be thousands of feet in thickness. Available geologic mapping by the United States Geological Survey (USGS) indicates the site is underlain by Holocene age alluvial fan deposits. Artificial fills from past episodes of site development and demolition are also present.

4. SEISMICITY AND GEOLOGIC HAZARDS

Geologists and seismologists recognize the San Francisco Bay Area as one of the most seismically-active regions in the United States. The significant earthquakes that occur in the Bay Area are associated with crustal movements along well-defined active fault zones that generally trend in a northwesterly direction.

The site and greater Bay Area are seismically dominated by the presence of the active San Andreas Fault System. In the theory of plate tectonics, the San Andreas Fault System is a transform fault that forms the boundary between the northward moving Pacific Plate (west of the fault) and the southward moving North American Plate (east of the fault). Locally, the movement is distributed across a complex system of strike-slip, right lateral parallel and subparallel faults, which include the San Andreas, Hayward and Calaveras faults, among others.

The table below presents approximate distances to active faults in the site vicinity based on web-based mapping by the California Geological Survey (CGS), as presented in an online fault database maintained by Caltrans. Site coordinates are N 38.2457°, W 122.0572°.

TABLE 4.1
REGIONAL FAULT SUMMARY

Fault Name	Approximate Distance to Site (miles)	Maximum Earthquake Magnitude, M_w
Cordelia	4 ½	6.7
Great Valley 5	5	6.9
Green Valley	5 ¾	6.8
Vaca	6½	6.7
Great Valley 04b	7¼	6.9
Los Medanos	11	6.8
Contra Costa Shear Zone	12	6.5
West Napa	12	7.0
Concord	13	6.6
Greenville	19¼	6.9
Hayward (Northern Extension)	23¼	7.3

Faults tabulated above and many others in the Bay Area are sources of potential ground motion. However, earthquakes that might occur on other faults within the northern California area are also potential generators of significant ground motion and could cause ground shaking at the site.

4.1 Surface Fault Rupture

The site is not within a currently established State of California Earthquake Fault Zone for surface fault rupture hazards. No active or potentially-active faults are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. By CGS definition, an active fault is one with surface displacement within the last 11,000 years. A potentially-active fault has demonstrated evidence of surface displacement with the past 1.6 million years. Faults that have not moved in the last 1.6 million years are typically considered inactive.

4.2 Ground Shaking

We used the USGS web-based *Unified Hazard Tool* to estimate the peak ground acceleration (PGA) and mean and modal magnitude associated with a 2,475-year return period that corresponds to an event with 2 percent chance of exceedance in 50 years. The USGS estimated PGA is 0.75 g and the mode (most probable) magnitude is 6.5 for Seismic Site Class D (V_s30 = 259 m/sec) based on a recent 2014 model within the application.

While listing PGA is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including frequency and duration of motion and soil conditions underlying the site.

4.3 Liquefaction

The site is not located within a State of California Seismic Hazard Zone for liquefaction since no such zones have been established in Solano County. Interactive web-based mapping by USGS indicates the site soils possesses a "high" susceptibility to liquefaction. Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary loss of shear strength due to pore pressure buildup under the cyclic shear stresses associated with intense earthquakes. Primary factors that trigger liquefaction are: moderate to strong ground shaking (seismic source), relatively clean, loose granular soils (primarily poorly graded sands and silty sands), and saturated soil conditions (shallow groundwater). Due to the increasing overburden pressure with depth, liquefaction of granular soils is generally limited to the upper 50 feet of a soil profile.

We assessed the potential for liquefaction using the computer software program *CLiq* (Version 2.2.0.35, Geologismiki) and the in-situ soil parameters measured in the CPT soundings. The software applied the methodology of Boulanger and Idriss (2014) to the CPT data to evaluate liquefaction potential and estimate resultant settlements. Our analysis considered the potential for cyclic softening in clayey soils and incorporated an earthquake moment magnitude (M_w) of 6.5 and a groundwater depth of 5 feet. Based on USGS seismic design criteria for 2019 CBC, a ground motion/Peak Ground Acceleration (PGA) of 0.73g was used in our analysis.

Our liquefaction analysis identified potentially liquefiable layers at each CPT location. In general, these layers are located more than 9 feet below existing grade. Consequences of liquefaction can include ground surface settlement, ground loss (sand boils) and lateral slope displacements (lateral spreading). For liquefaction-induced sand boils or fissures to occur, pore water pressure induced within liquefied strata must exert enough force to break through overlying, non-liquefiable layers. Based on methodology recommended by Youd and Garris (1995), which advanced original research by Ishihara (1985), a capping layer of non-liquefiable soil can prevent the occurrence of sand boils and fissures. Based on the presence of the non-liquefiable layer that mantles the site and the depth to significant liquefiable layers, the potential for ground loss due to sand boils or fissures in a seismic event is considered low.

Based on the depth to potentially liquefiable layers and the generally flat topography in the site vicinity, the potential for lateral spreading is considered low.

The likely consequence of potential liquefaction at the site is settlement. Our analysis indicates that total ground surface settlements up to approximately 1 inch may result from liquefaction and/or cyclic softening after a design-level seismic event. We recommend that foundations be designed to accommodate approximately ½ inch of differential seismic settlement across a horizontal distance of 50 feet. Output from our liquefaction analysis is presented in Appendix D.

4.4 Landslides

There are no known landslides near the site, nor is the site in the path of any known or potential landslides. We do not consider the potential for a landslide to be a hazard to this project.

4.5 Tsunamis and Seiches

The site is not located within a coastal area. Therefore, tsunamis (seismic sea waves) are not considered a significant hazard at the site.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Flooding from a seismically-induced seiche is considered unlikely.

5. SOIL AND GROUNDWATER CONDITIONS

5.1 Artificial Fill

Undocumented artificial fills were encountered in most of our borings and test pits to depths ranging from 2 ½ to 16 feet below existing grade. The deeper (16-foot-thick) fills were encountered within the former reservoirs and consisted of medium stiff clay and loose to medium dense gravel and sand with various amounts of debris. The debris commonly included broken concrete up to 5 feet in nominal dimension and lesser occurrences of asphalt chunks up to 3 feet in nominal dimension, brick fragments, rebar, broken pipes, rope, and fabric. The debris with our exploratory tests pits appears generally consistent with that reported in a previous environmental study by others. The artificial fills at the site are not suitable for the support of foundations loads such as those for the new multi-family buildings. Mitigation of the existing fills will be required.

Based on our laboratory test results, the clays within the artificial fills possess borderline medium to high plasticity and should be considered moderately expansive.

5.2 Alluvium

Outside of or below the artificial fills, our explorations encountered Holocene age alluvium consisting of medium stiff to hard clays with various amount of sand and gravel and medium dense clayey sand with variable amounts of gravel. We encountered alluvium to the maximum depth explored – approximately 50 feet below existing grade. Based on our laboratory test results, some clays within the native alluvium possess borderline medium to high plasticity and should be considered moderately expansive.

5.3 Groundwater

Groundwater was encountered at depths of approximately 6½ to 8½ feet below existing grade in our soil boring locations B1 through B4. Seepage was observed in our test pits at depths of approximately 5½ feet below the existing surface grades or deeper. Pore pressure measurements taken in our CPTs inferred groundwater depths of approximately 3 to 7 feet below the existing surface grades. Groundwater was encountered at depths of approximately 4½ feet in previous soil borings performed by others in 1989. Actual groundwater levels will fluctuate seasonally and with variations in rainfall, temperature and other factors and may be higher or lower than observed during our study.

5.4 Soil Corrosion Screening

Soil samples obtained during our field exploration were subjected to laboratory testing for minimum resistivity, pH, and chloride and water-soluble sulfate. We performed four soil corrosion potential screening by conducting laboratory testing on a representative near-surface soil sample. The laboratory test results and published screening levels are presented in Appendix B. Soil corrosivity should be considered in the design of buried metal pipes, underground structures, etc.

Water-soluble sulfate test results on selected samples of site soils indicate an S0 exposure classification for sulfate attack on normal portland cement concrete (PCC) as defined in Chapter 318, Table 19.3.1.1 of the ACI *Building Code Requirements for Structural Concrete.* ACI does not set forth requirements for S0 sulfate exposure classification. In addition, none of the soil samples tested would be classified as corrosive to buried metal improvements based on Caltrans criteria.

Geocon does not practice in the field of corrosion engineering and mitigation. If corrosion sensitive improvements are planned, it is recommended that a corrosion engineer be retained to evaluate corrosion test results and incorporate the necessary precautions to avoid premature corrosion of buried metal pipes and concrete structures in direct contact with the soils.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

6.1.1 Primary geotechnical constraints to site development are the presence of undocumented fill, shallow groundwater, and the presence of expansive soils. All undocumented fills should be removed and replaced with engineered fill in areas to support buildings or settlement sensitive improvements.

6.1.2 As discussed above, up to 16 feet of undocumented fill was encountered within the backfilled former water reservoirs. Based on our observations during field exploration, the reservoir fills are not suitable for the support of foundations loads such as those for the new multi-family buildings. Mitigation of the existing fills will be required. We have identified two mitigation alternatives that may be considered in project planning. Either of these alternatives would enable the use of conventional shallow foundations for the planned multifamily buildings.

<u>Alternative No. 1</u>

We recommend the existing undocumented fill be over-excavated to expose competent native soil or intact reservoir bottoms or sidewalls. The over-excavation within reservoir backfills should extend at least 25 feet beyond the perimeter of proposed structures and should be backfilled with properly compacted engineered fill. The over-excavated fills may be reused as an engineered fill provided the debris within is removed. Concrete blocks and other oversize material may be placed at depth within the engineered fill at the discretion and recommendation of the geotechnical engineer.

For buildings that straddle the limits of the former reservoirs, the portions of the building footprint outside the reservoir backfill should be over-excavated to a depth of 5 feet below existing or proposed subgrade, whichever is deeper, or to the depth necessary to expose competent native alluvium. Over-excavations in these areas should extend to at least 5 feet beyond the perimeter of proposed structures.

A temporary dewatering system will be necessary to implement this mitigation alternative. Design, selection of the equipment and dewatering method, and construction of temporary dewatering should be the responsibility of the contractor.

<u>Alternative No. 2</u>

The proposed buildings may be supported on a shallow foundation system if a ground improvement program is implemented to address the former reservoir fills. In addition to footing areas, we recommend that ground improvement be considered in slab areas to mitigate the potential settlement to levels acceptable to the owner and project designers. Future settlement in slab areas not supported by ground improvement could require corrective measures. A specialty ground improvement designer/contractor should be consulted to review appropriate ground improvement alternatives. The presence of debris within the fill material will pose challenges but preliminary input from ground improvement designers indicates that Drilled Displacement Columns (DDCs) may be feasible at this site (see Section 6.8).

For buildings that straddle the limits of the former reservoirs, the portions of the building footprint outside the reservoir backfill should be over-excavated to a depth of 2 feet below existing or proposed subgrade, whichever is deeper, or to the depth necessary to expose competent native alluvium. Over-excavations in these areas should extend to at least 5 feet beyond the perimeter of proposed structures.

The project team should review the information provided herein and other non-geotechnical factors when selecting foundation type or ground improvement for the project. Local agencies may not permit the use of ground improvement techniques that potentially allow the vertical migration of groundwater. The design of specialty foundation types or ground improvement systems should be reviewed by Geocon.

6.1.3 As discussed in Section 4.3, the site is susceptible to liquefaction. Our analysis indicates that, if liquefaction and/or cyclic softening were to occur, total ground surface settlements would be approximately 1 inch or less. We recommend the project be designed to accommodate at least ½ inch of seismically-induced settlement over a distance of 50 feet.

6.1.4 The proposed project redevelops a site with past episodes of grading and construction. As such, unknown underground improvements and areas of undocumented fill materials (not discussed herein) may be present. If encountered, supplemental recommendations will be provided during site development.

6.1.5 Any changes in the design, location or elevation of the proposed improvements, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

6.1.6 All references to relative compaction and optimum moisture content in this report are based on ASTM D 1557 (latest edition).

6.2 Seismic Design Criteria

6.2.1 We understand that seismic structural design will be performed in accordance with the provisions of the 2019 CBC which is based on the American Society of Civil Engineers (ASCE) publication *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-16). We derived the following seismic design parameters using the web-based Structural Engineers Association of California application *U.S. Seismic Design Maps*. Results are summarized in Table 6.2.1. The values presented are for the risk-targeted maximum considered earthquake (MCE_R) and Seismic Risk Category II.

TABLE 6.2.1
2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2019 CBC Reference
Site Class	D	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.652g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.58g	Figure 1613.2.1(2)
Site Coefficient, F_A	1.0	Table 1613.2.3(1)
Site Coefficient, F_V	1.72*	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.652g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.997g*	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.102g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.665g*	Section 1613.2.4 (Eqn 16-39)

Note:
*Per Section 11.4.8 of ASCE/SEI 7-16, a ground motion hazard analysis shall be performed for projects for Site Class "E" sites with Ss greater than or equal to 1.0g and for Site Class "D" and "E" sites with S1 greater than 0.2g. Section 11.4.8 also provides exceptions where ground motion hazard analysis may be waived. Using the code based values presented in the table above, in lieu of a performing a ground motion hazard analysis, requires the exceptions outlined in ASCE 7-16 Section 11.4.8 be followed in project design.

6.2.2 Table 6.2.2 presents additional seismic design parameters for projects with Seismic Design Categories of D through F in accordance with ASCE 7-16 for the mapped maximum considered geometric mean (MCE_G).

TABLE 6.2.2
2019 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.666g	Figure 22-7
Site Coefficient, F_{PGA}	1.1	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.732g	Section 11.8.3 (Eq. 11.8-1)

6.2.3 Conformance to the criteria presented in Tables 6.2.1 and 6.2.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a maximum level earthquake occurs. The primary goal of seismic design is to protect life and not to avoid structural damage, since such design may be economically prohibitive.

6.3 Soil and Excavation Characteristics

6.3.1 The onsite soils might be excavated with moderate effort using conventional excavation equipment. Additional effort may be required for excavations in artificial fill materials. We anticipate excavations in the reservoir backfill areas will generate oversize construction debris and deleterious materials not suitable for reuse in engineered fills. Contractors should review the subsurface conditions in our logs prior to bidding and selecting construction equipment and methods.

6.3.2 Unknown or unanticipated constituents may exist, especially within areas of artificial fill. Any artificial fills encountered at the site are undocumented and may contain constituents not reported herein. Below-grade improvements associated with prior site development may also be present.

6.3.3 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable Occupational Safety and Health Administration (OSHA) rules and regulations to maintain safety and maintain the stability of adjacent existing improvements.

6.3.4 The existing soils encountered at the site should be considered "expansive" as defined by 2019 CBC. The recommendations of this report assume proposed foundation systems will derive support in properly compacted fills and/or competent alluvial soils.

6.4 Materials for Fill

6.4.1 Over-excavated fill materials may be reused as engineered fill provided they are cleaned of debris. Engineered fills should not contain deleterious materials, or cementations larger than 6 inches in maximum dimension. Excavated soils may be wet and require drying prior to use and engineered fill. Concrete blocks and other oversize material may be placed at depth within the engineered fill at the discretion and recommendation of the geotechnical engineer.

6.4.2 Import or low-expansive fill material should be primarily granular with a " low" expansion potential (Expansion Index less than 20), a Plasticity Index less than 15, be free of organic material and construction debris, and not contain rock larger than 6 inches in greatest dimension. It should be assumed that soils excavated from the site do not meet the requirements for low-expansive fill.

6.4.3 Environmental characteristics and corrosion potential of import soil materials may also be considered. Proposed import materials should be sampled, tested, and approved by Geocon prior to its transportation to the site.

6.5 Grading

6.5.1 All clearing operations and earthwork (including over-excavation, scarification, and re-compaction) should be observed and all fills tested for recommended compaction and moisture content by representatives of Geocon.

6.5.2 Structural areas should be considered as areas extending a minimum of 5 feet horizontally from a foundation or beyond the outside dimensions of buildings, including footings and overhangs carrying structural loads, and where not restricted by property boundaries.

6.5.3 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor, civil engineer and geotechnical engineer in attendance. Special soil handling requirements can be discussed at that time.

6.5.4 Existing soils within building pads that are located wholly outside of the limits for the former reservoirs should be over-excavated to at least 2 feet below existing or proposed grade, whichever is lower. Deeper over-excavations may be needed to remove undocumented artificial fills.

6.5.5 After complete demolition and removal of existing structures, site preparation should commence with the removal of all existing improvements from the area to be developed/graded. All active or inactive utilities within the construction area should be protected, relocated, or abandoned. Any pipelines to be abandoned that are greater than 2 inches and less than 18 inches in diameter should be removed or filled with sand-cement slurry. Utilities larger than 18 inches in diameter should be removed. Excavations or depressions resulting from demolition and site clearing operations, or other existing excavations or depressions, should be restored with engineered fill in accordance with the recommendations of this report.

6.5.6 The exposed bottom surfaces and bottom processing should be observed by our representatives on a full-time basis. Supplemental recommendations may be provided based on site conditions during grading. Deeper over-excavations may be needed in some areas.

6.5.7 The upper 12 inches of pavement subgrade should be scarified and reworked, moisture conditioned to at least 2% above optimum and compacted to at least 92% relative compaction. (Prior to placing aggregate base, the finished subgrade should be proof-rolled with a laden water truck (or similar equipment with high contact pressure) to verify stability.

6.5.8 If grading commences in winter or spring, or in periods of precipitation, excavated and in-place soils may be wet. Earthwork contractors should be aware of potential compaction/workability difficulties. The most effective site preparation alternatives will depend on site conditions prior to and during grading operations; we should evaluate site conditions at those times and provide supplemental recommendations, if necessary.

6.5.9 All structural fill and backfill should be placed in layers no thicker than will allow for adequate bonding and compaction (typically 8 inches). Fill soils should be placed and compacted to at least 90% relative compaction for the upper 5 feet and at least 95% relative compaction below a depth of 5 feet (i.e., deeper excavation areas). The moisture content should be at least 2% above optimum moisture content (near optimum moisture where fill materials are predominantly sands or gravels).

6.6 Shallow Foundations

6.6.1 Shallow foundations (footings) founded in competent native soil or engineered fill, or supported by ground improvement, may be used for the planned residential buildings and for ancillary site structures such as short retaining walls, screen walls, or trash enclosures. The following recommendations assume that soils within 5 feet of finish grade will consist of moderately expansive materials.

6.6.2 It is recommended that isolated column spread footings should be at least 4 feet square and founded at least 24 inches below lowest adjacent pad grade, which is not to be confused with finished floor elevation. Column footings at the perimeter should be integral with the strip footing. The conventional continuous/strip footings have a minimum embedment depth of 24 inches below lowest adjacent pad grade and should be at least 12 inches wide.

6.6.3 Footings proportioned as recommended may be designed for an allowable soil bearing pressure of 3,000 pounds per square foot (psf). The allowable bearing pressure is for dead + live loads may be increased by up to one-third for transient loads due to wind or seismic forces.

6.6.4 The allowable passive pressure used to resist lateral movement of the footings may be assumed to be equal to a fluid weighing 300 pounds per cubic foot (pcf). The allowable coefficient of friction to resist sliding is 0.30 for concrete against soil. Combined passive resistance and friction may be utilized for design provided that the frictional resistance is reduced by 50%.

6.6.5 Minimum reinforcement for continuous footings should consist of four No. 5 steel reinforcing bars; two placed near the top of the footing and two near the bottom. Reinforcement for isolated spread footings should be determined by the structural engineer.

6.6.6 The foundation dimensions and minimum reinforcement recommendations presented herein are based upon soil conditions only and are not intended to be in lieu of those required for structural purposes.

6.6.7 Underground utilities running parallel to footings should not be constructed in the zone of influence of footings. The zone of influence may be taken to be the area beneath the footing and within a 1:1 plane extending out and down from the bottom edge of the footing.

6.6.8 The foundation subgrade should be sprinkled as necessary to maintain a moist condition without significant shrinkage cracks as would be expected in any concrete placement. Our representative should observe all footing excavations prior to placing reinforcing steel.

6.6.9 Where shallow foundation systems are designed and constructed as recommended herein, post-construction settlement due to dead + live loads should be approximately 1 inch or less with differential settlements of less than ½ inch across a horizontal distance of 50 feet.

6.7 Post-Tensioned Foundations

6.7.1 Post-tensioned foundations may be used to support the proposed residential structures and should be designed by a structural engineer experienced in post-tensioned slab design and design criteria of the Post-Tensioning Institute (PTI), Third Edition. The post-tensioned design should incorporate the geotechnical parameters presented on the table below. The parameters presented are based on the guidelines presented in the PTI, Third Edition design manual.

TABLE 6.7
POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS

Post-Tensioning Institute (PTI), Third Edition Design Parameters	Recommended Value
Equilibrium Suction	3.6
Edge Lift Moisture Variation Distance, eM (feet)	5.1
Edge Lift, yM (inches)	1.10
Center Lift Moisture Variation Distance, eM (feet)	9.0
Center Lift, yM (inches)	0.47

6.7.2 To reduce potential differential movement, all post-tensioned mats should be designed for an average mat contact pressure of 400 psf for dead plus live loads; at column or wall loading, the maximum localized bearing pressure should be limited to 2,500 psf.

6.7.3 Post-tensioned foundations should be embedded in accordance with the recommendations of the structural engineer. If a post-tensioned mat foundation system is planned, the slab should possess a thickened edge with a minimum width of 12 inches. The thickened edge should extend below the crushed rock underlayment layer.

6.7.4 The thickness of post-tensioned foundation systems should be determined by the project structural engineer. Based on our experience with similar projects and soils conditions, we anticipate the post-tensioned slab thicknesses will be on the order of 10 to 12 inches.

6.7.5 Our experience indicates that post-tensioned slabs are susceptible to excessive edge lift, regardless of the underlying soil conditions. Placing reinforcing steel at the bottom of the perimeter footings and the interior stiffener beams may mitigate this potential. Current PTI design procedures primarily address the potential center lift of slabs but, because of the placement of the reinforcing tendons in the top of the slab, the resulting eccentricity after tensioning reduces the ability of the system to mitigate edge lift. The structural engineer should design the foundation system to reduce the potential of edge lift occurring for the proposed structures.

6.7.6 During the construction of the post-tension foundation system, the concrete should be placed monolithically. Under no circumstances should cold joints be allowed to form between the footings/grade beams and the slab during the construction of the post-tension foundation system.

6.7.7 The use of isolated footings, which are located beyond the perimeter of the building and support structural elements connected to the building, are not recommended. Where this condition cannot be avoided, the isolated footings should be connected and tied to the building foundation system with grade beams.

6.7.8 Consideration should be given to connecting patio slabs to the building foundation to reduce the potential for future separation to occur.

6.7.9 Post-tensioned slabs should be underlain by at least 3 inches of ½-inch or ¾-inch crushed rock with no more than 5 percent passing the No. 200 sieve to serve as a capillary break.

6.7.10 Subgrade for post-tensioned foundations should be tested immediately prior to placing underlayment materials (crushed rock and vapor barrier) to verify that subgrade moisture content is appropriate.

6.7.11 Where post-tensioned foundation systems are designed and constructed as recommended herein, post-construction settlement due to dead + live loads should be approximately ¾ inch or less with differential settlements of less than ½ inch across a horizontal distance of 50 feet.

6.8 Ground Improvement

6.8.1 As an alternative to deep excavation stated in Section 6.1.2, the new buildings may be supported by a shallow foundation system if a ground improvement program is implemented to address the former reservoir fills. DDCs and similar systems increase density and lateral stress in the surrounding soil, claim improvement in bearing capacity and settlement potential, and mitigate the potential for liquefaction-induced settlements. DDC systems use high torque and heavy crowd equipment to construct pressure-grouted columns for increased foundation support characteristics. DDC systems also improve site soils through partial or full soil displacement resulting from cavity expansion during installation. DDC ground improvement systems are typically installed with tools 24 inches in diameter or less.

6.8.2 Other ground improvement systems may be feasible and the selected system may depend on non-geotechnical aspects such as the environmental characteristics of the soils that underlie the site. Given the composition of the fill material within the former reservoirs, costs for disposal of spoils and debris materials should be considered.

6.8.3 The ground improvement systems discussed herein typically allow the use of increased allowable bearing pressures for foundation design and result in estimated post-construction total settlement on the order of ¾ inch or less. Allowable bearing pressures would be provided by the ground improvement designer. Based on our prior experience, allowable bearing pressures of 6 kips per square foot may be assumed for planning purposes.

6.8.4 The specialty contractor should provide a complete design-build submittal with design calculations, engineered plans and specifications. Geocon should perform a geotechnical review of the ground improvement design.

6.8.5 Geocon should monitor all ground improvement construction. Our Quality Assurance (QA) services will supplement the contractor internal Quality Control (QC) program. Together the QA/QC program will monitor construction details such as drill depths, shaft length, average lift thicknesses, installation procedures, aggregate quality, and densification of lifts, as applicable. The allowable vertical capacities should be verified by full-scale modulus and uplift load tests performed on ground improvement elements. The contractor QC program should document each element installed, which will be reviewed by Geocon.

6.9 Concrete Slabs-on-Grade

6.9.1 Concrete slabs-on-grade subject to vehicle loading are considered pavements should be designed in accordance with the recommendations in Section 6.14 of this report.

6.9.2 Concrete slabs-on-grade for building structures, not subject to vehicle loading, should be a minimum of 5 inches thick and should be underlain by at least 12 inches of low-expansive fill meeting the requirements of Section 6.4.2 to reduce the potential for slab distress due to shrink/swell in the expansive soils. Minimum slab reinforcement should consist of No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Steel reinforcing should be positioned vertically near the slab midpoint. The low-expansive fill layer is not required if post-tensioned foundations systems are used.

6.9.3 Interior slabs or mat slabs in areas where moisture would be objectionable should be underlain by 3 inches of ½-inch or ¾-inch crushed rock with no more than 5% passing the No. 200 sieve to serve as a capillary break. The 3 inches of crushed rock should not be counted toward the 12 inches of low-expansive fill recommended above.

6.9.4 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. We recommend that at least 6 inches of Class 2 Aggregate Base (AB) compacted to at least 95% relative compaction be used below exterior concrete slabs. Prior to placing AB, the subgrade should be moisture conditioned to at least 2% over optimum and properly compacted to at least 90% relative compaction.

6.9.5 In lieu of specific recommendations from the structural or civil engineer, we recommend that crack control joints be spaced at intervals not greater than 8 feet for 4-inch-thick slabs (10 feet for 5-inch slabs). Crack control joints should extend a minimum depth of one-fourth the slab thickness and should be constructed using saw-cuts or other methods as soon as practical after concrete placement. Construction joints should be designed by the project structural engineer.

6.9.6 The recommendations of this report are intended to reduce the potential for cracking of slabs due to soil movement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to soil movement. This is common for project areas that contain expansive soils since designing to eliminate potential soil movement is cost prohibitive. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

6.10 Moisture Protection Considerations

6.10.1 A vapor barrier is not required beneath slab-on-grade for geotechnical purposes. Further, the migration of moisture through concrete slabs or moisture otherwise released from slabs is not a geotechnical issue. However, for the convenience of the owner, we are providing the following general suggestions for consideration by the owner, architect, structural engineer, and contractor. The suggested procedures may reduce the potential for moisture-related floor covering failures on concrete slabs-on-grade, but moisture problems may still occur even if the procedures are followed. If more detailed recommendations are desired, we recommend consulting a specialist in this field.

6.10.2 A vapor barrier meeting ASTM E 1745-09 Class C requirements may be placed directly below the slab, without a sand cushion. To reduce the potential for punctures, a higher quality vapor barrier (15 mil, Class A or B) should be used. The vapor barrier, if used, should extend to the edges of the slab, and should be sealed at all seams and penetrations.

6.10.3 The concrete water/cement ratio should be as low as possible. The water/cement ratio should not exceed 0.45 for concrete placed directly on the vapor barrier. Midrange plasticizers could be used to facilitate concrete placement and workability.

6.10.4 Proper finishing, curing, and moisture vapor emission testing should be performed in accordance with the latest guidelines provided by the American Concrete Institute, Portland Cement Association, and ASTM.

6.11 Temporary Excavations

6.11.1 The native alluvium can be considered a Type B soil in accordance with OSHA guidelines. Where free water, sandy or cohesionless soils or undocumented fills are encountered the materials should be downgraded to Type C. The contractor should have a "competent person" as defined by OSHA evaluate all excavations. All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping and possibly shoring.

6.11.2 It is the contractor's responsibility to provide sufficient and safe excavation support as well as protecting nearby utilities, structures, and other improvements which may be damaged by earth movements.

6.12 Retaining Wall Design

6.12.1 Lateral earth pressures may be used in the design of retaining walls and buried structures. Lateral earth pressures against these facilities may be assumed to be equal to the pressure exerted by an equivalent fluid. The unit weight of the equivalent fluid depends on the design conditions. Table 6.12 summarizes the weights of the equivalent fluid based on the different design conditions.

TABLE 6.12
RECOMMENDED LATERAL EARTH PRESSURES

Condition	Equivalent Fluid Density
Active	55 pcf
At-Rest	75 pcf

6.12.2 Unrestrained walls should be designed using the active case. Unrestrained walls are those that are allowed to rotate more than 0.01H (where H is the height of the wall). The above soil pressures assume level backfill under drained conditions within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall and no surcharges within that same area.

6.12.3 Unless project-specific loading information is provided by the structural engineer, where vehicle loads are expected atop the wall backfill, an additional uniform surcharge pressure equivalent to 2 feet of backfill soil should be used for design. Where the vehicle loading will be limited to passenger cars, the additional uniform surcharge equivalent may be reduced to 1 foot of backfill soil.

6.12.4 Retaining walls greater than 2 feet tall (retained height) should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and should be waterproofed as required by the project architect. Positive drainage for retaining walls should consist of a vertical layer of permeable material positioned between the retaining wall and the soil backfill. The permeable material may be composed of a composite drainage geosynthetic or a natural permeable material such as crushed gravel at least 12 inches thick and capped with at least 12 inches of native soil. A geosynthetic filter fabric should be placed between the gravel and the soil backfill. Provisions for removal of collected water should be provided for either system by installing a perforated drainage pipe along the bottom of the permeable material which leads to suitable drainage facilities.

6.12.5 We recommend that all retaining wall designs be reviewed by Geocon to confirm the incorporation of the recommendations provided herein. In particular, potential surcharges from adjacent structures and other improvements should be reviewed by Geocon.

6.13 Underground Utilities

6.13.1 Underground utility trenches should be backfilled with properly compacted material. The material excavated from the trenches should be adequate for use as backfill provided it does not contain deleterious matter, vegetation or rock larger than six inches in maximum dimension. Trench backfill should be placed in loose lifts not exceeding eight inches and should be compacted to at least 90% relative compaction at least 2% above optimum moisture content (near optimum where backfill materials are predominantly sands and gravels).

6.13.2 Bedding and pipe zone backfill typically extends from the bottom of the trench excavations to a minimum of 6 inches above the crown of the pipe. Pipe bedding material should consist of crushed aggregate, clean sand or similar open-graded material. Proposed bedding and pipe zone materials should be reviewed by Geocon prior to construction; open-graded materials such as ¾ inch drain rock may require wrapping with filter fabric to mitigate the potential for piping. Pipe bedding and backfill should also conform to the requirements of the governing utility agency.

6.13.3 Utility trenches backfilled with granular material (including pipe bedding material) may serve as conduits for groundwater and may cause pumping, seepage or other undesirable effects at the lower ends of trench lines. Consideration should be given to constructing “trench plugs” at periodic intervals along utility line alignments to reduce those potential problems. Trench plugs should be located where the utility trench enters the perimeter of a structural area. Trench plugs may consist of compacted native clay soil or concrete. Trench plug material should completely surround the pipe and be in contact with the undisturbed walls and bottom of the trench. The length of soil trench plugs should be on the order of one to two feet. The geotechnical engineer should review the placement and design of trench plugs prior to plan finalization.

6.14 Pavements

6.14.1 The upper 12 inches of pavement subgrade should be scarified and reworked, moisture conditioned to at least 2% above optimum and compacted to at least 95% relative compaction. (Prior to placing aggregate base, the finished subgrade should be proof-rolled with a laden water truck (or similar equipment with high contact pressure) to verify stability.

6.14.2 We recommend the following asphalt concrete (AC) pavement sections for design to establish subgrade elevations in pavement areas. The project civil engineer should determine the appropriate Traffic Index (TI) based on anticipated traffic conditions. The flexible pavement sections below are based on estimated design TIs. We can provide additional sections based on other TIs if necessary.

TABLE 6.14
FLEXIBLE PAVEMENT SECTION RECOMMENDATIONS

Location	Estimated Traffic Index (TI)	AC (inches)	AB (inches)
Parking Stalls	4.5	3	8
Driveways	6.0	3 ½	12 ½
Heavy Duty	7.0	4	15 ½

Note: The recommended flexible pavement sections are based on the following assumptions:

1. Subgrade soil has an R-Value of 5.
2. AB: Class 2 AB with a minimum R-Value of 78 and meeting the requirements of Section 26 of the latest Caltrans Standard Specifications.
3. AB is compacted to 95% or higher relative compaction at or near optimum moisture content. Prior to placing AB, the subgrade should be proof-rolled with a loaded water truck to verify stability.
4. AC: Asphalt concrete conforming to local agency standards or Section 39 of the latest Caltrans Standard Specifications.

6.14.3 The AC sections in Table 6.14 are final, minimum thicknesses. If staged-pavements are used, the construction bottom AC lift should be at least 2 inches thick. Following construction, the finish top AC lift should be at least 1½ inches thick.

6.14.4 Unless specifically designed and evaluated by the project structural engineer, where concrete paving will be utilized for support of vehicles, we recommend the concrete be a minimum of 6 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. In addition, doweling, reinforcing steel or other load-transfer mechanism should be provided at joints if desired to reduce the potential for vertical offset. The concrete should have a minimum 28-day compressive strength of 3,500 psi.

6.14.5 We recommend that at least 6 inches of Class 2 Aggregate Base (AB) be used below rigid concrete pavements. The aggregate base should be compacted to at least 95% relative compaction near optimum moisture content.

6.14.6 Consideration should be given to providing a thickened edge on the outside of concrete slabs subject to wheel loads. The thickened edge should be 2 inches thicker than the design slab thickness at the slab edge and taper back to the design slab thickness 3 feet behind the face of the slab.

6.14.7 In general, we recommend that concrete pavements be designed, constructed and maintained in accordance with industry standards such as those provided by the American Concrete Pavement Association.

6.14.8 Crack control joints should be spaced at intervals not greater than 12 feet for 6-inch-thick slabs (10 feet for 5-inch slabs and 8 feet for 4-inch slabs) and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness and should be constructed using saw-cuts or other methods as soon as practical after concrete placement. Construction joints should be designed by the project structural engineer.

6.14.9 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 6 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving. Alternatives such as plastic moisture cut-offs or modified drop-inlets may also be considered in lieu of deepened curbs.

6.14.10 The asphalt pavement section recommendations herein are based on the design procedures of Caltrans Highway Design Manual (HDM). It should be noted that most rational pavement design procedures are based on projected street or highway traffic conditions and may not be representative of vehicular loading that occurs in parking lots and driveways. Pavement proximity to landscape irrigation, reduced traffic speed and short turning radii increase the potential for pavement distress to occur in parking lots even though the volume of traffic is significantly less than that of an adjacent street. The HDM indicates that the resulting pavement sections for parking lots are minimized to keep initial costs down but are reasonable because additional AC surfacing can be added later, if needed, and generally without incurring traffic hazards or traffic handling problems. It is generally not economically feasible to design and construct the entire parking lot and driveways for the unique loading conditions previously described. Periodic maintenance of the pavement in these areas, therefore, should be anticipated.

6.15 Surface Drainage

6.15.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change to important engineering properties. Proper drainage should be maintained at all times.

6.15.2 All site drainage should be collected and transferred to the street in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundations or retaining walls. Drainage should not be allowed to flow uncontrolled over any descending slope. The proposed structures should be provided with roof gutters. Discharge from downspouts, roof drains and scuppers not permitted onto unprotected soils within five feet of the building perimeter. Planters which are located adjacent to foundations should be sealed or properly drained to prevent moisture intrusion into the materials providing foundation support. Landscape irrigation within five feet of the building perimeter footings should be kept to a minimum to just support vegetative life.

6.15.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. The building pad and pavement areas should be fine graded such that water is not allowed to pond. Final soil grade should slope a minimum of 2% away from structures.

6.15.4 We recommend implemented measures to reduce infiltrating surface water near buildings and slabs-on-grade. Such measures may include:

- Selecting drought-tolerant plants that require little or no irrigation, especially within 5 feet of buildings, slabs-on-grade, or pavements.
- Using drip irrigation or low-output sprinklers.
- Using automatic timers for irrigation systems.
- Appropriately spaced area drains.
- Hard-piping roof downspouts to appropriate collection facilities.

7. FURTHER GEOTECHNICAL SERVICES

7.1 Plan and Specification Review

7.1.1 We should review project plans and specifications prior to final design submittal to assess whether our recommendations have been properly implemented and evaluate if additional analysis and/or recommendations are required.

7.2 Testing and Observation Services

7.2.1 The recommendations provided in this report are based on the assumption that we will continue as Geotechnical Engineer of Record throughout the construction phase and provide compaction testing and observation services and foundation observations throughout the project. It is important to maintain continuity of geotechnical interpretation and confirm that field conditions encountered are similar to those anticipated during design. If we are not retained for these services, we cannot assume any responsibility for others interpretation of our recommendations, and therefore the future performance of the project.

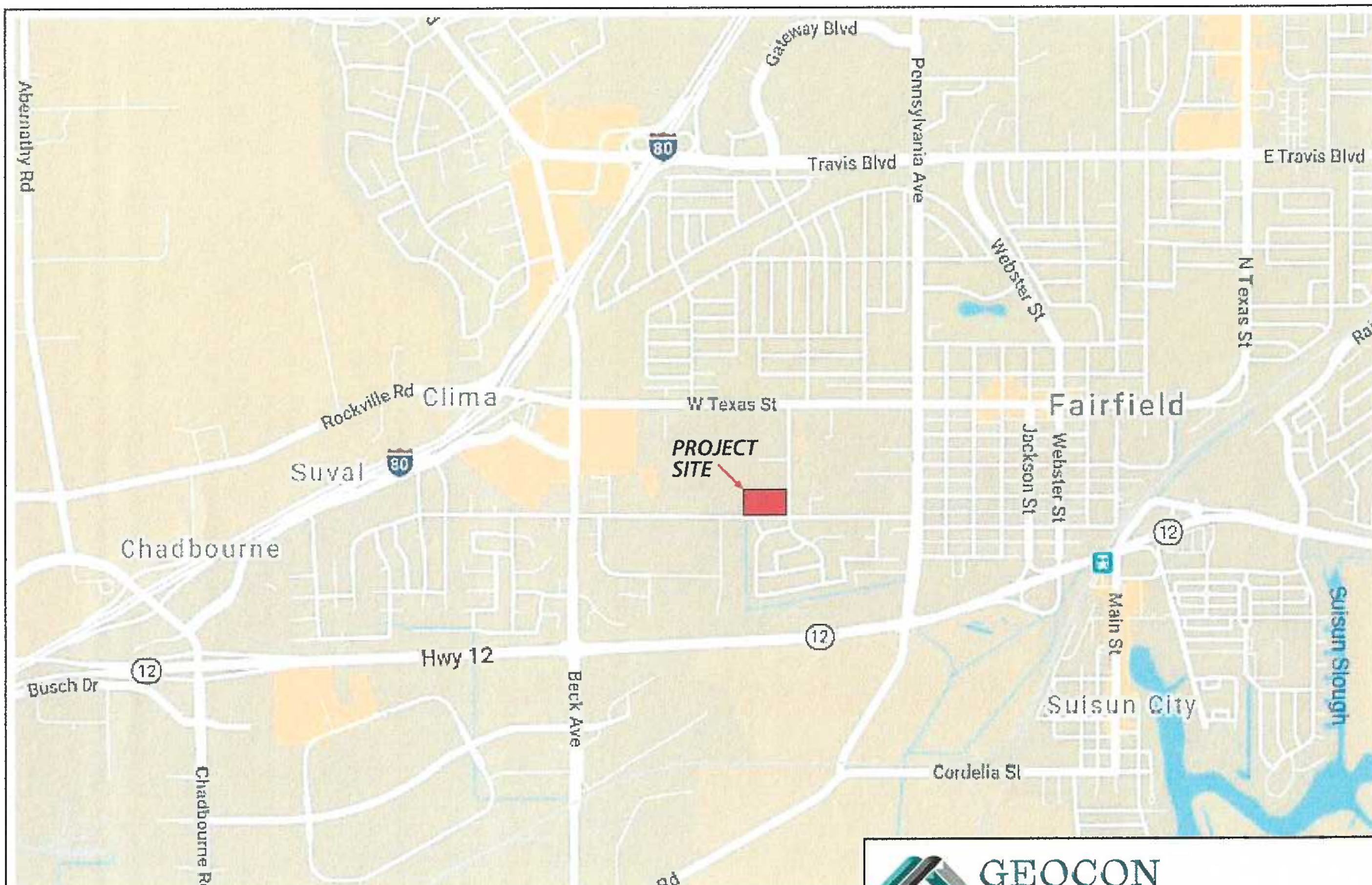
LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Consultants, Inc. should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the geotechnical scope of services provided by Geocon Consultants, Inc.

This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices used in the site area at this time. No warranty is provided, express or implied.



0 1/2
Scale in Miles

N

GEOCON
CONSULTANTS, INC.
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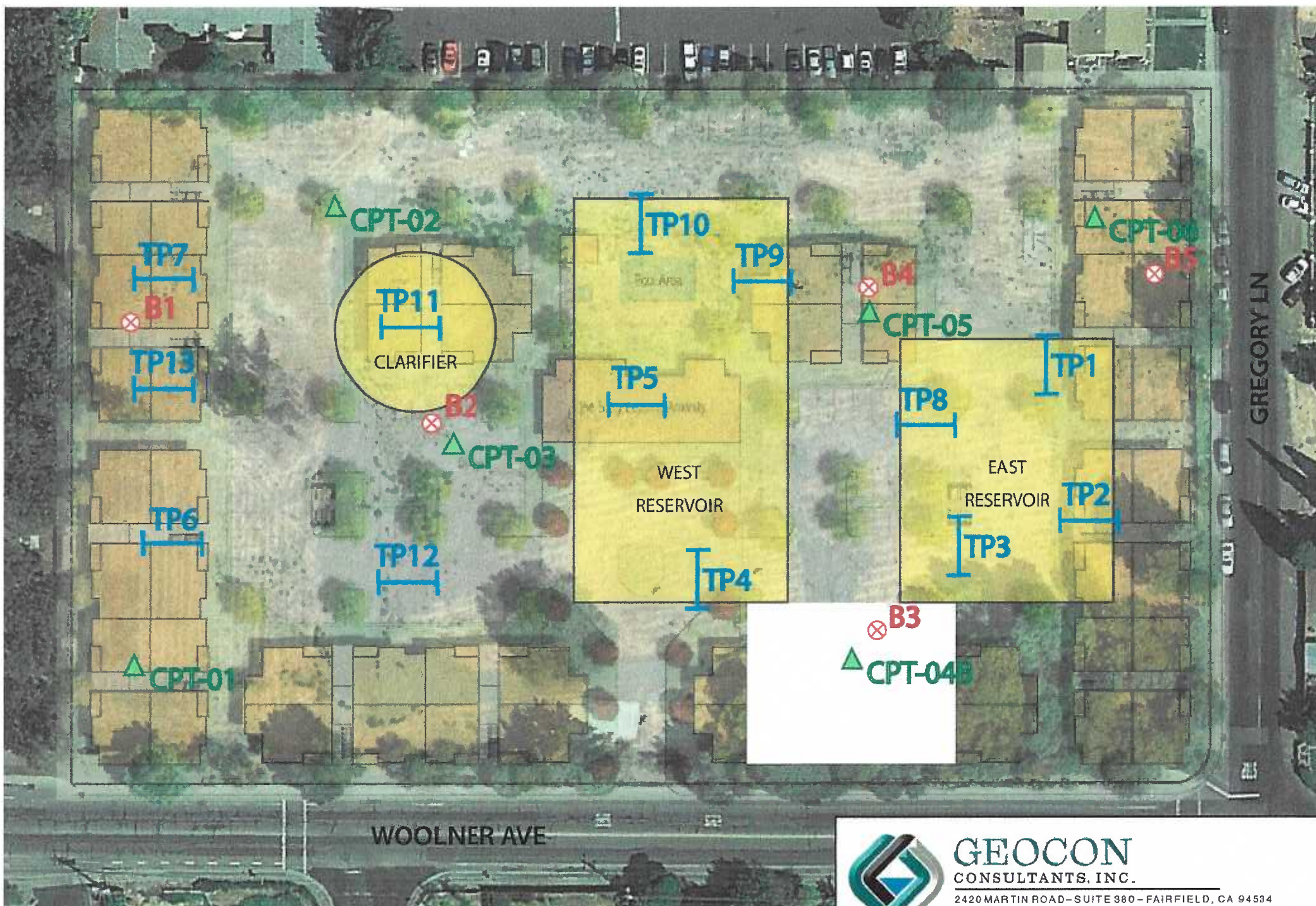
PHONE 925.961.5271 - FAX 925.371.5915

Woolner-Gregory MF Residential

Northwest Corner of Woolner Avenue & Gregory Lane
Fairfield, California

VICINITY MAP

E9186-04-01	June 2020	Figure 1



Legend:

- ⊗ = Approximate Boring Location
- △ = Approximate CPT Location
- ├─┤ = Approximate Test Pit Location

0 100
Scale in Feet

N

Note: CPT-04B was initially attempted at its original location, but twice encountered practical refusal due to obstruction and moved westward by 5 feet each time.

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Woolner-Gregory MF Residential		
Northwest Corner of Woolner Avenue & Gregory Lane Fairfield, California		
SITE PLAN		
E9186-04-01	June 2020	Figure 2

EXHIBIT "B"

INSTRUCTIONS TO WITHHOLD FUNDS AFTER CLOSE OF ESCROW (REPAIRS)

(Attached.)

PLACER TITLE COMPANY
A MOTHER LODE COMPANY

INSTRUCTIONS TO WITHHOLD FUNDS AFTER CLOSE OF ESCROW (REPAIRS)

643084-1004-0

DATE: December 14, 2020

TO: Placer Title Company, ESCROW HOLDER

RE: ESCROW NO. : P-381893

You are hereby instructed to withhold the sum of $400,000.00 from funds due the Seller herein at the close of your above numbered escrow for the following purpose:

Repairs specified as follows: Soils mitigation work

We the undersigned, buyers, hereby accept the estimate for work from - ____________________(Contractor) and acknowledge receipt ofthe same. Upon receipt by Placer Title Company of WRITTEN INSTRUCTIONS FROM SELLER AND BUYER AND RECEIPT OF INVOICES FROM THE ABOVE NAMED CONTRACTOR AND/OR COMPANY, you are authorized and instructed to pay said invoice. No more than 1 request for payment shall be submitted every 30 days.

Escrow Holder shall not hold the above funds or any portion thereof, in excess of 180 days beyond the Close of Escrow date. If no written instructions have been received from the above authorized persons, Escrow Holder shall have the right to interplead the funds as set forth in the General Provisions, paragraph 10.

Any excess funds are to be returned to the Seller. In the event the costs exceed the amount set forth above, Escrow Holder is to disburse only the amount held and shall not be responsible or assume any liability for said excess. Buyer and Seller understand that the funds held hereunder are solely for the purpose of payment of the work set forth in these instructions.

The parties hereto acknowledge and agree that Escrow Holder shall **NOT** accept any appended instructions from any party with respect to this instruction after the close of escrow, but shall disburse funds held solely in accordance with the joint instructions as they exist at the close of escrow.

BUYERS:	SELLERS:
Sutton Place Development Corp., a California corporation	Fairfield Housing Authority, a municipal corporation
By: ______________________ Patrick Morrill, Pres.	By: ______________________ Stefan Chatwin, Executive Director
Affordable Housing Alliance II, Inc., a Colorado nonprofit corporation	
By: ______________________ Philip Wood, Pres.	
Date:______________	Date:______________

PLACER TITLE COMPANY
A MOTHER LODE COMPANY

SUPPLEMENTAL ESCROW INSTRUCTIONS

TO: Placer Title Company
1300 Oliver Rd., Suite 120
Fairfield, CA 94534

Escrow No. P-381893

Escrow Officer: Kelly Guglielmo

Date: December 14, 2020

RE: Release of funds held

Property Address: 1600 Woolner, Fairfield, CA 94533

You are hereby instructed to release funds which were held in escrow as follows:

Pay to:
Pay to:
Pay to:
Pay to:

Buyer:

Sutton Place Development Corp., a California corporation

By: ________________________________
Patrick Morrill, Pres.

Affordable Housing Alliance II, Inc., a Colorado nonprofit corporation

By: ________________________________
Philip Wood, Pres.

Seller:

Fairfield Housing Authority, a municipal corporation

By: ________________________________
Stefan Chatwin, Executive Director

Forwarding Address:

____

____

EXHIBIT "C"

SCHEDULE OF PERFORMANCE

This Schedule of Performance requires the submission of plans or other documents at specific times. Some of the submissions are not described in the text of the Agreement. Such plans or other documents, as submitted, must be complete and adequate for review by the FHA and the City of Fairfield or other applicable governmental entity when submitted. Prior to the time set forth for each particular submission, the Developer shall consult with FHA staff informally as necessary concerning such submission in order to assure that such submission will be complete and in a proper form within the time for submission set forth herein.

Action	Date / Deadline
Items 1 – 10 Relate to Developer Actions and Requirements Prior to or through/at the Close of Escrow	
1. Delivery of Amendment to Escrow Holder. The Parties shall deliver a copy of the executed First Amendment to DDA to Escrow Holder.	Within five (5) business days after the date of the First Amendment to DDA. (The parties confirm that escrow has already been opened and a copy of the executed DDA has been delivered to Escrow Holder.)
2. Project Budget; Equity; (The Project Budget is attached as Exhibit "F".) Developer shall submit a schedule of sources and uses of funds, with reasonable evidence of required equity.	Prior and as a condition to the Close of Escrow.
3. Preliminary Plans. Developer shall submit preliminary Plans and Specifications to the City of Fairfield and to the FHA	Not later than sixty (60) days after the date of the First Amendment to DDA
4. Design Development Plans. Developer shall submit interim "design development" Plans and Specifications to the City of Fairfield and to the FHA.	Not later than ninety (90) following the delivery by City of Fairfield of the City of Fairfield's and FHA's comments to the Preliminary Plans.
5. Final Plans and Specifications. The Developer shall submit the Final Plans and Specifications for City of Fairfield and FHA approval.	Prior and as a condition to the Close of Escrow.
6. Building Permits. The Building Permits for the construction of the Improvements are capable of being issued, subject to postponement of fees (that must be paid at the Close of Escrow through escrow if not paid earlier).	Condition to the Close of Escrow.
7. Performance and Payment Bonds. The Developer shall deliver to the FHA copies of any required performance and payment bonds per Section 2.4.6.	Prior and as a condition to the Close of Escrow.
8. Insurance. The Developer shall submit evidence of insurance to the FHA.	Prior and as a condition to the Close of Escrow.
Items 9 – 15 Relate to the Conveyance of the Land and Developer Actions and Requirements After the Close of Escrow	

Action	Date / Deadline
9. Close of Escrow. The Developer shall purchase the Land from the FHA.	On or before February 10, 2022.
10. Construction Schedule	Prior to and as a condition to the Close of Escrow.
11.	
12.	

EXHIBIT "D"

SCOPE OF DEVELOPMENT

168 affordable rental units (all restricted by tax credit regulatory agreement or alternative regulatory agreement), 3-story garden style building(s) with property management office, swimming pool, barbecues and outdoor dining area, fitness center, indoor and outdoor community space, tot lot, surface parking areas, and gardens.

EXHIBIT "F"

PROJECT BUDGET

(Attached.)

PARKSIDE FLATS - FAIRFIELD, CA (Project No. 213)
PRO FORMA INCOME AND EXPENSE SUMMARY

168 UNIT FAMILY APTS. - TAX EXEMPT BOND FINANCING W/ 4% LIHTC CREDITS

UNIT MIX & RENTAL REVENUE SUMMARY

UNITS						PER UNIT					
						RESIDENT PORTION					
Total	Affordable	UNIT TYPE	UNIT SQUARE FOOTAGE	TOTAL SQUARE FOOTAGE	RESTRICTED MEDIAN INCOME PERCENTAGE	GROSS MONTHLY RENT	UTILITY ALLOWANCE	NET MONTHLY RENT	HOUSING SUBSIDY RENTS (net of Util. Allow.)	Addt'l Revenue - Per Unit Housing Subsidy Assistance	Total Project Monthly Rental Income (excl. Housing Subsidy)
4	4	1 BD - 1 BA - Flat	644	2,576	30.00%	$ 520	(34)	$ 486	$ -	$ -	$ 1,945
5	5	1 BD - 1 BA - Flat	644	3,220	50.00%	867	(34)	833	-	-	4,165
16	16	1 BD - 1 BA - Flat	644	10,304	60.00%	1,040	(34)	1,006	-	-	16,102
17	17	1 BD - 1 BA - Flat	644	10,948	70.00%	1,214	(34)	1,180	-	-	20,057
4	4	2 BD - 2 BA - Flat	878	3,512	30.00%	625	(47)	578	-	-	2,312
5	5	2 BD - 2 BA - Flat	878	4,390	50.00%	1,041	(47)	995	-	-	4,973
16	16	2 BD - 2 BA - Flat	878	14,048	60.00%	1,249	(47)	1,203	-	-	19,243
17	17	2 BD - 2 BA - Flat	878	14,926	70.00%	1,457	(47)	1,411	-	-	23,985
4	4	2 BD - 2 BA - Flat	908	3,632	30.00%	625	(47)	578	-	-	2,312
4	4	2 BD - 2 BA - Flat	908	3,632	50.00%	1,041	(47)	995	-	-	3,978
16	16	2 BD - 2 BA - Flat	908	14,528	60.00%	1,249	(47)	1,203	-	-	19,243
16	16	2 BD - 2 BA - Flat	908	14,528	70.00%	1,457	(47)	1,411	-	-	22,574
5	5	3 BD - 2 BA - Flat	1,094	5,470	30.00%	721	(58)	663	-	-	3,316
3	3	3 BD - 2 BA - Flat	1,094	3,282	50.00%	1,202	(58)	1,144	-	-	3,432
16	16	3 BD - 2 BA - Flat	1,094	17,504	60.00%	1,442	(58)	1,384	-	-	22,150
18	18	3 BD - 2 BA - Flat	1,094	19,692	70.00%	1,683	(58)	1,625	-	-	29,246
2	0	2 BD - 2 BA - Mgr	908	1,816	70.00%	-	-	-	-	-	-
Total Unit Square Footage				148,008							
Add: Office, Community Room & Laundry Facilities				29,602							
Add: Residential Unit Gross SF. Adjustment				-							
Add: Other - Common Area Hallways				4,000							-
168	166			181,610							$ 199,039

ANNUAL INCOME, EXPENSE AND RESERVES

				ANNUAL TOTALS	PER UNIT ANNUAL AVERAGE
INCOME					
GROSS RENTAL INCOME				$ 2,388,466	$ 14,217
OPERATING SUBSIDIES					
Other Project Based Rental Subsidy Reserve			$ -	-	-
OTHER INCOME:					
Miscellaneous	$ 12.00	/unit/mo.	$ 24,192		
Other -	$ -	/unit/mo.	-	24,192	144
TOTAL GROSS POTENTIAL INCOME				2,412,658	14,217
VACANCY, BAD DEBT, & CONCESSIONS ALLOWANCE					
Gross Rental Income	5.00%			(119,423)	(711)
Operating Subsidies	5.00%			-	-
Other Income	5.00%			(1,210)	(7)
EFFECTIVE GROSS INCOME (EGI)				2,292,025	13,499
OPERATING EXPENSES					
Residential Operating Expenses					
Personnel Costs				-	-
Administration/General				-	-
Marketing Expense				-	-
Repairs & Maintenance				-	-
Cleaning & Decorating				-	-
Contract Services				-	-
Utilities - Electric				-	-
Insurance				-	-
Miscellaneous				681,119	4,054
Management Fee	(4.00% annually; $45.48 /u/mo.)			91,681	546
Total Residential Operating Expenses				772,800	4,600
Other Operating Expense					
Real estate taxes (Voter Indebtedness & assessments		Annual Escalation	2.00%	2,500	15
Other - CalHFA MIP Administrative Fee			$ 7,500	7,500	45
Residential Services	$ -	per Unit		60,000	357
Total Other Operating Expenses				70,000	417
TOTAL OPERATING EXPENSES				842,800	5,017
NET OPERATING INCOME				$ 1,449,225	$ 8,482
RESERVES					
REPLACEMENT RESERVES (unit/year)	$ 300.00	Annual Escalation	3.00%	$ 50,400	$ 300
NET CASH FLOW FROM OPERATIONS, AFTER RESERVES				**$ 1,398,825**	$ 8,182

PARKSIDE FLATS - FAIRFIELD, CA (Project No. 213)

SOURCES AND USES - DEVELOPMENT COSTS

168 UNIT FAMILY APTS. - TAX EXEMPT BOND FINANCING W/ 4% LIHTC CREDITS

SOURCES OF FUNDS		
CONSTRUCTION PERIOD (All Sources Prior to Permanent Loan Funding)		
CONSTRUCTION LOAN		$ 51,296,262
CAPITAL CONTRIBUTION - Limited Partner - Equity Investor		5,719,068
CONST. PERIOD HOLDBACKS (disbursed upon funding of or after Perm. Loan)		8,261,183
CASH FLOW FROM OPERATIONS - Pre-Perm. Loan Funding		832,350
TOTAL CONSTRUCTION PERIOD SOURCES		$ 66,108,862
PERMANENT SOURCES:		
CAPITAL CONTRIBUTION - Limited Partner (Tax Credit Equity)		$ 28,595,340
CASH FLOW FROM OPERATIONS - Pre-Perm. Loan Funding		832,350
PERM. LOAN NO. 1 - CONVENTIONAL PERM. LOAN		22,890,000
REFUNDABLE DEPOSITS		184,660
DEVELOPER FEE NOTE - DEVELOPER		4,713,512
CALHFA MIP LOAN		8,893,000
CITY LOAN		-
TOTAL PERMANENT SOURCES		$ 66,108,862
USES OF FUNDS		
ACQUISITION AND PRE-DEVELOPMENT		
PURCHASE PRICE - LAND	$ 1,100,000	
DEMOLITION	500,000	
LEGAL - Acquisition	-	
CLOSING & TITLE	30,000	$ 1,630,000
DIRECT COSTS		
OFFSITE IMPROVEMENTS	500,000	
ONSITE IMPROVEMENTS	857,261	
BASE CONSTRUCTION - RESIDENTIAL	31,940,000	
GENERAL REQUIREMENTS	1,689,863	
CONTRACTOR OVERHEAD AND PROFIT	1,749,356	36,736,480
INDIRECT COSTS		
ARCHITECTURE	500,000	
ENGINEERING	375,000	
FEES AND PERMITS	8,400,000	
FEASIBILITY	175,000	
LEGAL AND ACCOUNTING	195,000	
MISC. ORG. / TAXES / INSUR.	255,000	
FURNISHINGS AND FIXTURES	300,000	
MARKETING AND LEASING	200,000	
DEVELOPER FEE	7,809,011	18,209,011
FINANCING COSTS		
TAX CREDIT APPLICATION / MONITORING FEES	108,780	
REFUNDABLE TCAC/CDLAC Deposits	184,660	
LOAN FINANCING COSTS - Pre-Construction Loan	30,000	
LOAN FINANCING COSTS - CalHFA MIP Loan	113,930	
LOAN FINANCING COSTS - Construction Loan	1,269,500	
LOAN FINANCING COSTS - Permanent Loan	343,900	
INTEREST EXPENSE – Construction Period Only	2,103,759	
INTEREST EXPENSE – Post Construction Period	2,158,271	
OPERATING RESERVE	1,029,507	7,383,534
CONTINGENCY		
CONTINGENCY - CONSTRUCTION	1,836,824	
CONTINGENCY - SOFT COSTS	313,013	2,149,837
TOTAL USES OF FUNDS		$ 66,108,862

PARKSIDE FLATS - FAIRFIELD, CA (Project No. 213)

PROJECT COSTS AND ELIGIBLE BASIS SUMMARY

168 UNIT FAMILY APTS. - TAX EXEMPT BOND FINANCING W/ 4% LIHTC CREDITS

	PROJECT COSTS	ELIGIBLE BASIS: Rehab and/or New Const. Costs	ELIGIBLE BASIS: Acquisition Credit Costs
Land Costs			
Purchase Price / Lease Incentive Payment	$ 1,100,000	$ xxxxxxxxx	$ xxxxxxxxx
Demolition / Landscaping	500,000	xxxxxxxxx	xxxxxxxxx
Legal / Broker Fees / Escrow & Title / Pre-Dev. Int. Carry	30,000	xxxxxxxxx	xxxxxxxxx
Total Land Costs	**1,630,000**	**-**	**-**
Total Acquisition Costs			
Off-Site Improvements	500,000	125,000	xxxxxxxxx
Total Acquisition Costs	**500,000**	**125,000**	**-**
Construction Costs			
Sitework - Onsite	857,261	857,261	xxxxxxxxx
Structures	31,940,000	31,940,000	-
General Requirements	1,689,863	1,689,863	xxxxxxxxx
Contractor Overhead	699,742	699,742	xxxxxxxxx
Contractor Profit	1,049,614	1,049,614	xxxxxxxxx
Bonds	-	-	xxxxxxxxx
General Liability Insurance	-	-	xxxxxxxxx
Total New Construction Costs	**36,236,480**	**36,236,480**	**-**
Architectural Costs			
Design	500,000	500,000	xxxxxxxxx
Total Archtectural Costs	**500,000**	**500,000**	**-**
Total Survey & Engineering Costs	**375,000**	**375,000**	**xxxxxxxxx**
Construction Period Interest, Fees & Other Costs			
Construction - Interest	2,103,759	2,103,759	xxxxxxxxx
Construction - Origination & Loan Fees	530,000	530,000	xxxxxxxxx
Construction - Bond Costs	614,500	614,500	xxxxxxxxx
Construction - Lender Inspection, Cost Review & Other	10,000	10,000	xxxxxxxxx
Construction - Title and Recording	35,000	35,000	xxxxxxxxx
Predevelopment Loan - Interest & Fees	41,227	41,227	xxxxxxxxx
Predevelopment Loan - Fees	30,000	30,000	xxxxxxxxx
Other Loans - Interest & Fees	113,930	113,930	xxxxxxxxx
Bond Premium	-	-	xxxxxxxxx
Property Taxes	20,000	20,000	xxxxxxxxx
Insurance	200,000	200,000	xxxxxxxxx
Total Acquis./Rehab. Per. Interest, Fees & Other Costs	**3,698,415**	**3,698,415**	**-**
Permanent Financing Costs			
Perm. Loan - Origination Fee	228,900	xxxxxxxxx	xxxxxxxxx
Perm. Loan - Credit Enhancement & Application Fee	15,000	xxxxxxxxx	xxxxxxxxx
Perm. Loan - Title and Recording	25,000	xxxxxxxxx	xxxxxxxxx
Perm Loan - Bond Costs	-	xxxxxxxxx	xxxxxxxxx
Other	25,000	xxxxxxxxx	xxxxxxxxx
Total Permanent Loan Financing Costs	**293,900**	**-**	**-**
Legal Fees & Professional (Exc. R.E. acquis. & synd. fees)			
Construction Loan - Lender Legal paid by Applicant	80,000	80,000	xxxxxxxxx
Construction Loan - Borrower Legal paid by Applicant	50,000	50,000	xxxxxxxxx
Perm. Loan - Lender Legal paid by Applicant	50,000	xxxxxxxxx	xxxxxxxxx
Perm. Loan - Borrower Legal paid by Applicant	25,000	xxxxxxxxx	xxxxxxxxx
Other - Legal (Incl. in Eligible Basis)	50,000	50,000	xxxxxxxxx
Total Legal & Professional Fees	**255,000**	**180,000**	**-**
Total Appraisal Costs	**10,000**	**10,000**	**xxxxxxxxx**
Reserves			
Post Construction Period Interest Reserve	2,158,271	xxxxxxxxx	xxxxxxxxx
Operating Reserve	1,029,507	xxxxxxxxx	xxxxxxxxx
Total Reserves	**3,187,778**	**-**	**-**
Contingency Costs			
Contingency - Hard Costs	1,836,824	1,836,824	xxxxxxxxx
Contingency - Soft Costs	313,013	313,013	xxxxxxxxx
Total Contingency Costs	**2,149,837**	**2,149,837**	**-**
Other Project Costs			
Tax Credit Agency / CDLAC Application & Allocation Fee	39,900	xxxxxxxxx	xxxxxxxxx
TCAC / CDLAC Refundable Deposit	184,660	xxxxxxxxx	xxxxxxxxx
Tax Credit Agency Monitoring Fee	68,880	xxxxxxxxx	xxxxxxxxx
Environmental Audit	30,000	30,000	xxxxxxxxx
Local Permits & Fees - Development Impact Fees	5,040,000	5,040,000	xxxxxxxxx
Local Permits & Fees - Permit Processing Fees	3,360,000	3,175,340	xxxxxxxxx
Accounting (Incl. Cost Certification)	70,000	70,000	xxxxxxxxx
Market Study	15,000	15,000	xxxxxxxxx
Feasibility	120,000	120,000	xxxxxxxxx
Marketing & Resident Service Setup	200,000	xxxxxxxxx	xxxxxxxxx
Furniture, Fixtures, & Equipment	300,000	300,000	xxxxxxxxx
Other Miscellaneous Project Costs	35,000	35,000	xxxxxxxxx
Total Other Project Costs	**9,463,440**	**8,785,340**	**-**
Developer Fee			
Developer Fee - Developer Portion	7,809,011	7,809,011	-
Total Developer Fees	**7,809,011**	**7,809,011**	**-**
TOTAL PROJECT COSTS	**$ 66,108,862**	**$ 59,869,084**	**$ -**