Exhibit F

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GMD Engineers
Foundation Engineering

SOIL INVESTIGATION REPORT (Design Phase)

Project Number: GMD 2019041

Project: New Single Family Dwelling with a Guest House

For: Craig Suhl

APN: 015-192-006-000

Location: 6235 Brookdale Dr Carmel CA 93923

11 West Laurel Dr. Suite #225 Salinas, CA 93906
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GMD Engineers
Foundation Engineering

December 1, 2019

Craig Suhl
6235 Brookdale Dr Carmel CA 93923

SUBJECT: SOIL INVESTIGATION REPORT Design Phase

Dear Craig Suhl,

In accordance with your authorization, GMD Engineers has completed a soil investigation report Design Phase for your proposed project located at 6235 Brookdale Dr Carmel CA 93923 (APN: 015-192-006-000) which we drilled/investigated on 12/01/2019.

This report includes the results of field and laboratory testing and recommendations for foundation design; as well as site development. It is our opinion that this site is suitable for the proposed development from soil engineering standpoint. The recommendations are based upon applicable standards at the time this report was done.

It has been a pleasure to be of service to you on this project. If you have any questions regarding the attached report, please don't hesitate to contact us at (831) 840-4284.

Respectfully Submitted,

GMD Engineers

REGISTERED PROFESSIONAL ENGINEER
GERONIMO MARTIN DALIVA
C-65185
CIVIL ENGINEER
STATE OF CALIFORNIA
ENGINEER LICENSE
EXPIRATION: 9-30-21

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This report has been prepared for the exclusive use of Craig Suhl at 6235 Brookdale Dr Carmel CA 93923 with specific application to the proposed project.

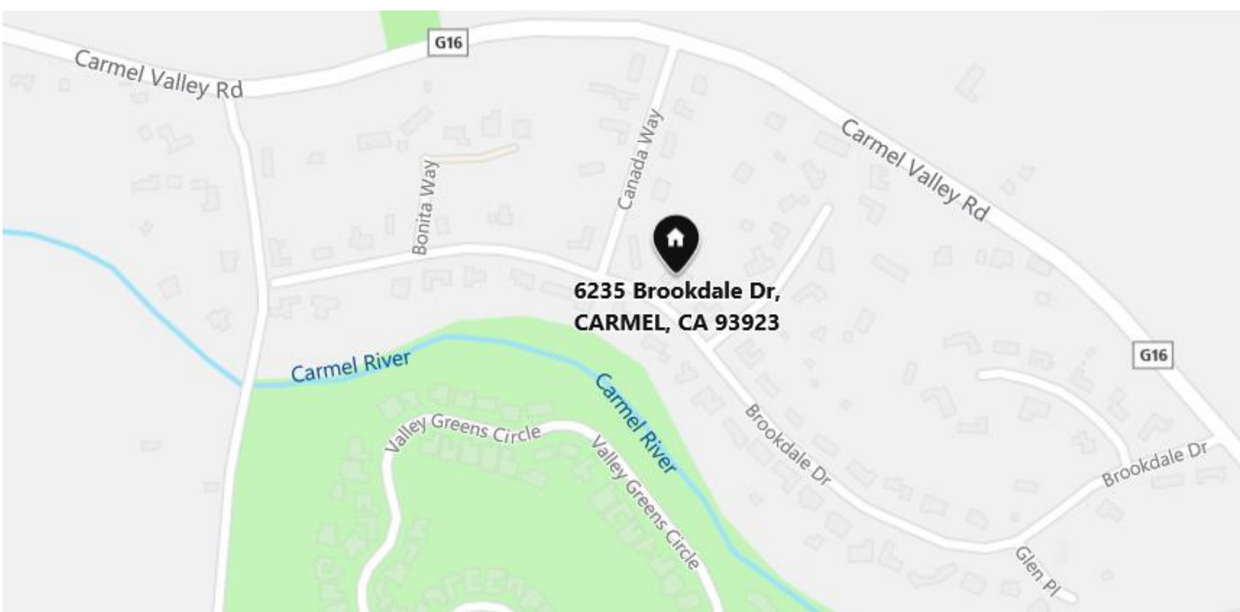
1 PROJECT DESCRIPTION

1.1 PROPOSED DEVELOPMENT

We understand that the proposed project will include single family dwelling.

This report was prepared for the exclusive use of our client and their consultants for design of this project. In the event of project change such as the locations and scope of work of the proposed structures, or any other site features change from what is shown on the site plan included in this report, GMD Engineers should be notified so that the changes can be reviewed to determine if the recommendations presented in this report are still applicable or whether modifications are necessary.

1.2 SITE DESCRIPTION



LOCATION MAP

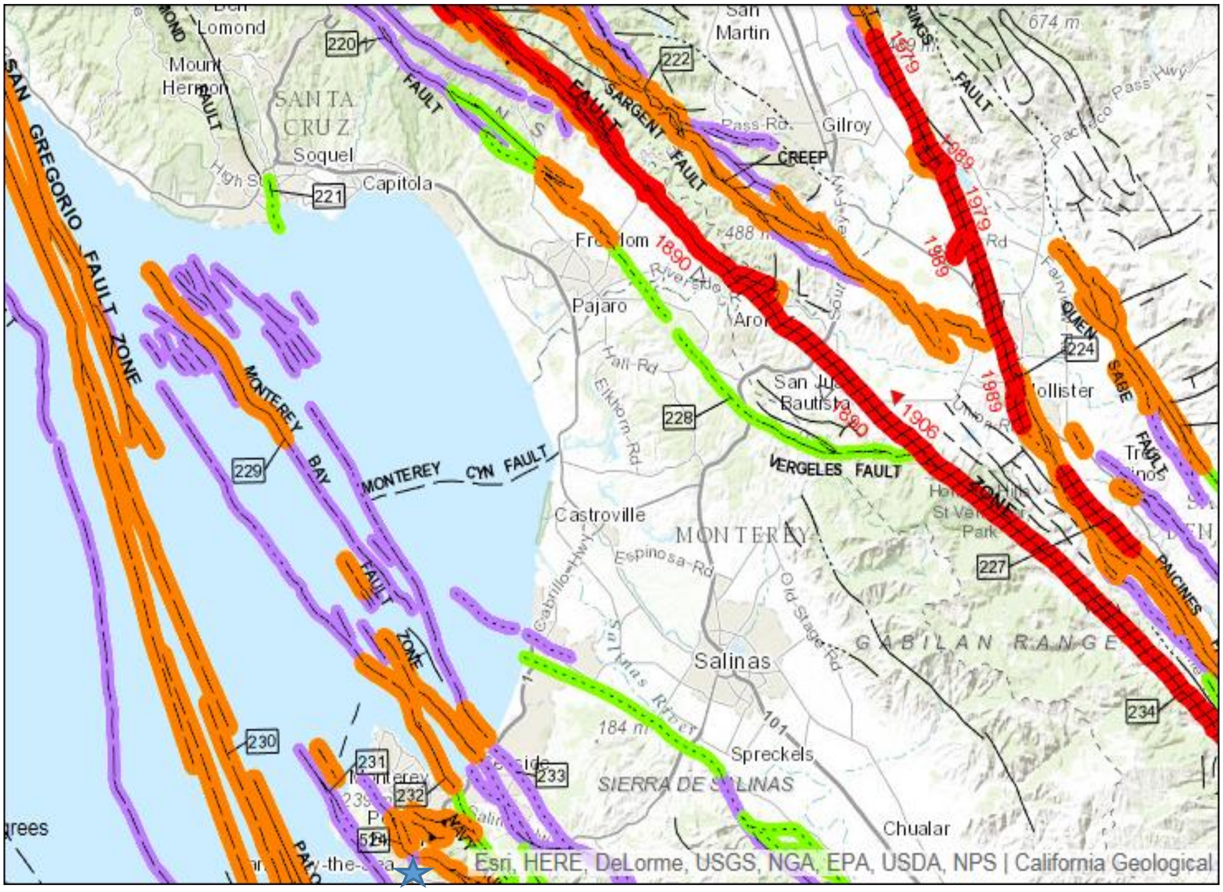
The site in which a 4-bed, 3 bath, one 1/2 bath, (3,033sq ft) house is located has an area of 42,000 sq ft. It is planned to demolish the existing house and rebuild a new single family dwelling. The property is predominantly flat.

1.3 GEOTECHNICAL SETTING

The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone and per the fault map below, no known surface expression of active faults is believed to exist within the site. California Central Coast is seismically active and the planning area can be expected to experience periodic minor earthquakes and possibly a major earthquake on one of the nearby active faults during the life of the proposed project. Upon review of the Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada, Monterey is traversed by: San Andreas Fault, 20 km from the site. Other faults that may cause very strong and violent ground shaking are: Berwick Canyon (reverse), Chupines (strike-slip), Cypress Point (reverse), Hatton Canyon (reverse), Laureles (reverse), Ord Terrace (reverse), Seaside (reverse), Sylvan thrust (reverse), Tularcitos/Navy/Monterey Bay (strike-slip) & Tularcitos/Navy/Monterey Bay (reverse). For each of the active faults, the distance from the planning area and estimated maximum moment magnitude are summarized in following table on regional faults & seismicity:

REGIONAL FAULTS & SEISMICITY

Fault Segment	Approximate Distance from Site (miles)	Direction from Site	Slip Rate	Maximum Characteristic Magnitude
San Andreas - 1906 Segment	40	Southwest	24	7.90



REGIONAL FAULT NEAR PROJECT SITE

Salinas City is a city in Monterey County, California. Monterey County is traversed by a number of both 'active" and 'potentially active" faults most of which are relatively minor hazards for the purposes of the site development. Monterey County, the entire mapped onshore active fault traces lie along the main San Andreas Fault. As such, this site will experience seismic activity of various magnitudes emanating from one or more of the numerous faults in the region. Although, fault rupture through the site, is not anticipated.

The **San Andreas Fault (Type A)** situated south-east of the subject is approximately 12 miles away. It is named after San Andreas Lake, a small body of water that was formed in a valley between the two plates, is a continental transform fault that extends roughly 1300 km (810 miles) through California. It forms the tectonic boundary between the Pacific Plate and the North American Plate, and its motion is right-lateral strike-slip (horizontal). The fault divides into three segments, each with different characteristics and a different degree of earthquake risk, the most significant being the southern segment, which passes within about 35 miles of Los Angeles. A 2015 study in partnership with the

U.S. Geological Survey predicted a 7% chance of a magnitude 8.0 earthquake along San Andreas Fault in the next 30 years. Some scientist calls such magnitude of earthquake, the next "Big One".

The two largest historically recent earthquakes on the San Andreas to affect the area were the moment magnitude (M_w) 7.9 San Francisco earthquake of April 1906 and the M_w 6.9 Loma Prieta earthquake of October 1989. The San Francisco earthquake caused severe seismic shaking and structural damage to many buildings in the Monterey Bay area.

Geologists have divided the San Andreas Fault system into segments with characteristic earthquakes of different magnitudes and recurrence intervals. The Working Group on Northern California Earthquake Potential (WGNCEP) in 1996 redefined the segments and characteristic earthquakes for the San Andreas Fault system. Two overlapping segments pose the greatest seismic hazard at the project site. The northern section represents the rupture along the San Andreas fault that occurred during the 1906 Mw 7.9 earthquake extending from Cape Mendocino to San Juan Bautista with a comparable magnitude earthquake recurrence interval of about 200 years. The second segment is known as the Santa Cruz Mountain segment and represents the rupture zone of the 1989 Mw 6.9 Loma Prieta earthquake with an independent recurrence interval of approximately 138 years.

The site is likely to be shaken by earthquakes of approximately 8.0 (similar to the “San Francisco earthquake of 1906, with an average recurrence between 138 to 188 years along North coast segment of San Andreas Fault. Also, earthquakes of magnitude 6 to 7 are likely along the faults within the San Mateo are.

MAJOR HISTORICAL EARTHQUAKES IN THE REGION

YEAR	EPICENTER	RICHTER MAGNITUDE AT EPICENTER
1901	Parkfield	6.4
1906	San Francisco	8.3
1922	Parkfield	6.3
6.6	Parkfield	6.0
1966	Parkfield	6.6
1983	Coalinga	6.5

1984	Morgan Hill	6.1
1989	Loma Prieta	7.1
2003	San Simeon	6.5
2004	Parkfield	6.0
2019	Ridgecrest	7.1

Source: U.S. Geological Survey 2019

While Richter magnitude provides a useful measure of comparison between earthquakes, the Moment magnitude is more widely used for scientific comparison since it accounts for the actual slip that generated the earthquake.

Actual damage is due to the propagation of seismic or ground waves from initial failure and the intensity of shaking are as much related to earthquake magnitude as the condition of underlying materials. Loose materials tend to amplify ground waves, while hard rock can quickly attenuate them, causing little damage to overlying structures. For this reason, the Modified Mercalli Intensity (MMI) Scale provides a useful qualitative assessment of earthquake intensity. The MMI Scale is shown in the table below.

Modified Mercalli Intensity (MMI) Scale

Mercalli Intensity	Equivalent Richter Magnitude	Witness Observations
I	1.0 to 2.0	Felt by very few people; barely noticeable.
II	2.0 to 3.0	Felt by a few people, especially on upper.
III	3.0 to 4.0	Noticeable indoors, especially on upper floors, but may not be recognized as an earthquake.
IV	4.0	Felt by many indoors, few outdoors. May feel like heavy truck passing by.
V	4.0 to 5.0	Felt by almost everyone, some people awakened. Small objects moved trees and poles may shake.
VI	5.0 to 6.0	Felt by everyone. Difficult to stand. Some heavy furniture moved, some plaster falls. Chimneys may be slightly damaged.

VII	6.0	Slight to moderate damage in well built, ordinary structures. Considerable damage to poorly built structures. Some walls may fall.
VIII	6.0 to 7.0	Little damage in specially built structures. Considerable damage to ordinary buildings, severe damage to poorly built structures. Some walls collapse.
IX	7.0	Considerable damage to specially built structures, buildings shifted off foundations. Ground cracked noticeably. Wholesale destruction. Landslides.
X	7.0 to 8.0	Most masonry and frame structures and their foundations destroyed. Ground badly cracked. Landslides. Wholesale destruction.
XI	8.0	Total damage. Few, if any, structures standing. Bridges destroyed. Cracks in ground. Waves seen on ground.
XII	8.0 or greater	Total damage. Waves seen on ground. Objects thrown up into air.

Source: Abridged from
The Severity of an Earthquake, USGS General Interest Publication.

1.3 GEOTECHNICAL & GEOLOGICAL SEISMIC HAZARDS

LANDSLIDING. Since the majority of the planning area is currently gently sloping, seismically induced landsliding within the planning area is considered low.

GROUND SURFACE FAULT RUPTURE. Ground surface fault rupture occurs along the surficial traces of active faults during significant seismic events. Due to the location of the nearest active or potentially active fault, the San Andreas Fault fault, which is mapped at approximately 10 miles from the project site, the potential for ground surface fault rupture is therefore considered high.

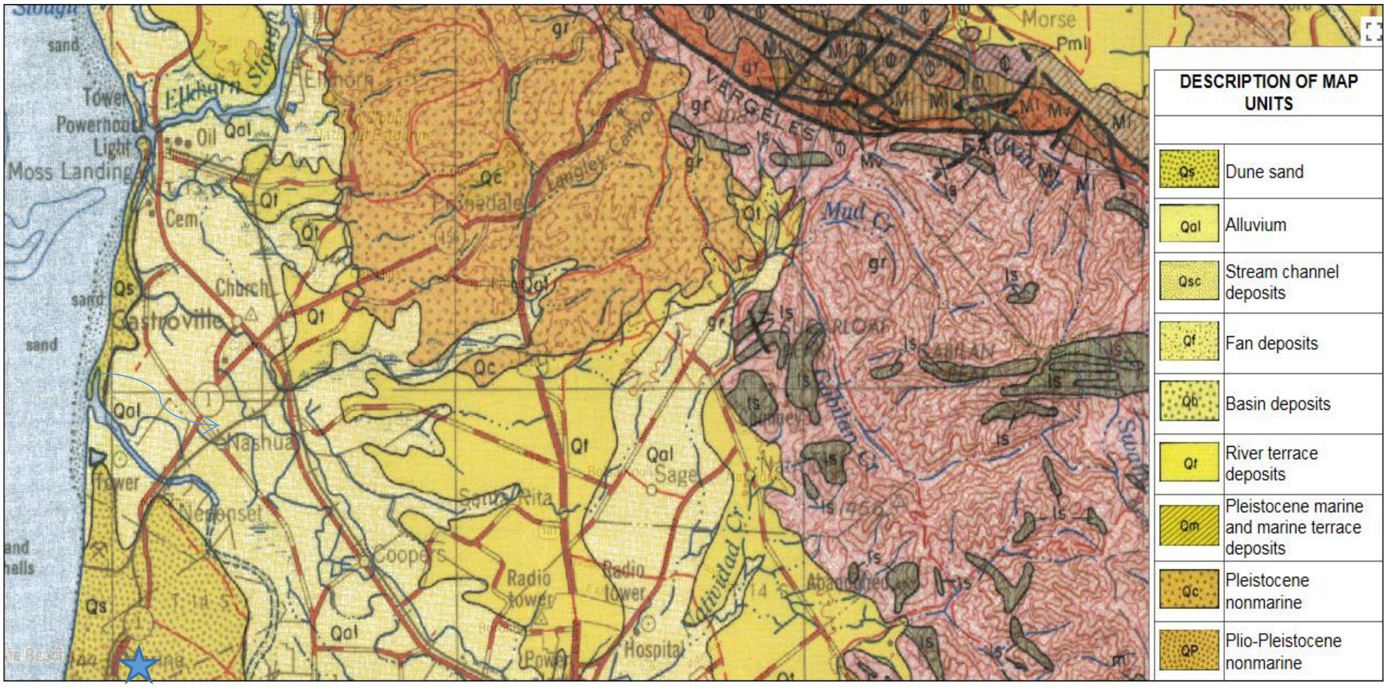
LIQUEFACTION AND LATERAL SPREADING HAZARDS. Liquefaction tends to occur in loose, saturated fine-grained soils, course silts or clays with low plasticity. The liquefaction process typically occurs at depths less than 50 feet below the ground surface, although liquefaction can occur at deeper intervals, given the right conditions. In order for liquefaction to occur there must be the proper soil type, soil saturation, and cyclic accelerations of sufficient magnitude to progressively increase the water pressures within the soil mass. Non-cohesive soil shear strength is developed by the point-to-point contact of the soil grains. As the water pressures increase in the void spaces surrounding the soil grains, the soil particles become supported more by the water than the point-to-point contact. When the water pressures increase sufficiently, the soil grains begin to lose contact with each other resulting in the loss of shear strength and continuous deformation of the soil where the soil begins to liquefy.

Liquefaction can lead to several types of ground failure, depending on slope conditions and the geological and hydrological settings, of which the four most common types of ground failure are: 1) lateral spreads, 2) flow failures, 3) ground oscillation and 4) loss of bearing strength. Based on our field investigations and laboratory testing, liquefaction is not anticipated as a seismic hazard. Therefore, the potential for lateral spreading in the project location is also considered low.

GROUND SHAKING. Intense ground shaking generated by earthquakes from nearby local faults will likely occur within the project site. Ground shaking within the planning area would depend on several factors including: the earthquake magnitude, distance of the epicenter, and subsurface conditions. The U.S. Geological Service has estimated that the San Andreas Fault could produce a maximum predicated earthquake of 8.5 on the Richter scale. Other faults in the area could produce a maximum of between

6.5 and 7.5. In these events, the potential for strong to severe ground shaking within the planning area would he high.

GEOLOGICAL MAP



REGIONAL GEOLOGY MAP NEAR PROJECT SITE

Site Geology:

General geologic features pertaining to the project site were evaluated by reference to Geologic Data Map No. 2 of the California Geological Survey (2010). Based on the publication, the project site and its vicinity is generally underlain by the following Quaternary geologic units:

Q - Pleistocene to Holocene alluvium, lake, playa, and terrace deposits; unconsolidated and semi-consolidated.
Qoa - Older Pleistocene to Holocene alluvium, lake, playa, and terrace deposits.

Legend:
Q - Alluvium, lake, playa, and terrace deposits.
Qoa - Older alluvium, lake, playa, and terrace deposits.
Source:
California Geological Survey (2010), Geologic Map of California, Geologic Data Map No. 2, Compilation and Interpretation by Jennings (1977).

2.0 INVESTIGATION AND TESTING

2.1 SUBSURFACE GEOTECHNICAL EXPLORATION

Based on our site and boring log investigation and exploration, the site soil properties indicate that the sub-surface on the site are relatively consistent, however, there are variations in color, moisture content, and density across the site.

The subsurface exploration portion of the investigation consisted of one (1) drill rig boring that were conducted under our observation on December 01, 2019 by California Geotech.

We observed drilling of one boring and logged the subsurface conditions eastern portion of the property. Boring location is shown on Site Plan, Appendix. We retained a portable drill rig and crew to advance the boring using 4-inch diameter solid flight auger methods.

Boring 1 was advanced to a depth of 5 feet below existing grade, then refusal. Boring were backfilled with drill cuttings. We obtained soil at 5 feet depth using standard penetration tests and a 2" O.D. split spoon SPT sampler. The blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The sampler was driven 18 inches and the number of blows were recorded for each 6 inches of penetration. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 1 foot of penetration; the blow counts have not been converted using any correction factors. When sampler driving was difficult, penetration was recorded only as inches penetrated for 50 hammer blows.

Soil samples were obtained at selected intervals in the soil test borings. All samples were identified according to project number, boring number and depth, encased in polyethylene plastic wrapping to protect against moisture loss, and transported to the laboratory in special containers.

The soil samples were labeled, photographed, wrapped up in transparent membrane and stored in 5-gal plastic containers according to their depth.

The following tests had been performed: moisture test (ASTM D2937-04) and D2216-05; a grain size distribution test (ASTM D 422-63 (2002); plasticity index test (ASTM D 4318-05).

We used the field log to develop the report logs in Appendix A. The log depicts subsurface conditions at the exploration locations for the date of exploration; however, subsurface conditions may vary with time.

2.2 EXPANSIVE NATURE OF THE SOIL

The surface soils are low to medium expansive characteristics.

1. Moisture condition soil to at least 4 percentage points over the optimum moisture content.
2. Wet with clean water the excavated foundation 24 hours before pouring of concrete

2.3 LIQUEFACTION POTENTIAL

There is no history of liquefaction at the site.

3.0 SUBSURFACE CONDITIONS

3.1 STRATIGRAPHY

The following soil types were encountered in the soil test borings performed at the site:

Boring 1 encountered one foot of 6 inches to 12 inches unsuitable materials, such as organic soils & roots. Below the unsuitable materials, 5-7 feet of dark brown clayey SAND, dense. Small units of fat clay continued below 18 ft.

Our laboratory testing indicates that this soil exhibits low to moderate shrink/swell potential with variations in moisture content.

Expansive soil can cause distress to foundations, floor slabs, pavements, sidewalks, and other improvements, which are sensitive to soil movements. We define expansive soil as any soil with a plasticity index greater than 15; soils with a plasticity index of less than 15 can be considered non-expansive.

The recommendations given in this report are such that settlements are negligible and as such are of little concern. The expected total settlement is expected to be ¼ inch and the expected differential settlement is less than ½ of that value.

Site Preparation

The project calls for an existing deck replacement. Concrete pavement, building rubble, concrete foundations and any other debris noted at or below the existing ground surface should be removed as part of the site preparation for the proposed construction area.

Excavations

Temporary construction slopes should be designed and excavated in strict compliance with the rules and regulations of the Federal Register, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA), 29 CFR, Part 1926. This document was prepared to better insure the safety of workers entering trenches or excavations, and requires that all excavations conform to the new OSHA guidelines.

The side walk of trenches constructed in these materials will be prone to sudden collapse (for trenches deeper than 2 feet) unless they are properly shored and braced or laid back at an appropriate angle. Project designers should make a clear note of this fact in the project specifications and on the project plans and should draw attention to contractor and particularly the underground contractor, to the property shore and brace or lay back the sides walls of trenches.

All work should comply with the State of California Construction Safety Orders for "Excavations, Trenches, and Earthworks".

For the purpose of this section of the report, utility pipes, free draining sand should be used as bedding. Sand bedding should be compacted to at least 90% relative compaction based on ASTM Test Procedure D 1557-00, or to the degree of compaction specified by the utility designer.

Detailed description of the type of soil layers encountered during drilling is given in the borehole logs (Appendix B). The lines designating the interface between soil strata on the boring logs represent approximate boundaries; transition between materials may be gradual.

4.0 GROUNDWATER

Groundwater was found not found during drilling. However, groundwater levels may fluctuate with seasonal climatic variations and changes in the land use. Low permeability soils will require several days or longer for groundwater to enter and stabilize in the test borings.

5.0 RECOMMENDATIONS

The recommendations presented in the following sections of this report are based on the information available regarding the proposed construction, the results obtained from our soil test borings and laboratory tests, and our experience with similar projects. Because the test borings represent a very small statistical sampling of subsurface conditions, it is possible that conditions may be encountered during construction that are substantially different from those indicated by the soil test borings. In these instances, adjustments to design and construction may be necessary.

Table 1. Seismic Design Parameters, 2019 CBC

Latitude, Longitude: 36.5377365, -121.86504480000002

S_s	S_1	Site Class	Fa	Fv	S_{MS}	S_{M1}	S_{DS}	S_{D1}	Occupancy Category	Seismic Design Category
1.264	0.471	D	1		1.264		0.842		II	D

The structure is placed in Seismic Design Category (SDC) **D**.

NOTE: Please refer to Appendix "C" for Seismic Parameters Calculations.

Expected Total and Differential Settlement.

The contractor is solely responsible for protecting excavations by shoring, sloping, benching or other means as required to maintain stability of both the excavation sides and bottom. GMD Engineers does not assume any responsibility for construction site safety or the activities of the contractor.

For this site, the overburden soil encountered in our exploratory borings consisted of mostly fat clay. We anticipate that OSHA will classify these materials as type B. OSHA recommends a maximum slope inclination of 1H: 1V for type B soils. Excavation requirements will vary depending on the actual soil conditions in some areas. Temporary construction slopes should be closely observed for signs of mass movement, such as tension cracks near the crest, bulging at the toe of the slope, etc.

Structural Fill

We do not anticipate structural fill in this project.

However, we recommend that structural fill and backfill, if any, be compacted in accordance with the criteria standard engineering practice. A qualified field representative should periodically observe fill placement operations and perform field density tests at various locations throughout each lift, including trench backfill, to indicate if the specified compaction is being achieved.

STANDARD STRUCTURAL FILL PLACEMENT GUIDELINES

Areas of Fill Placement	Compaction Recommendation (ASTM D1557-Standard Proctor)	Moisture Content (Percent of Optimum)
Granular cushion beneath Floor Slab and over Footings	90%	As necessary to obtain density
Structural fill supporting Footings	90%	-1 to +3 percent
Structural fill placed within 5 feet beyond the perimeter of the building pad	90%	-1 to +3 percent
Grade-raise fill placed within 1 foot of the base of the pavement	90%	-1 to +3 percent
Structural fill placed below the base of the Pavement Soil Sub grade	90%	-1 to +3 percent
Utility Trenches - Within building and pavement areas	90%	-1 to +3 percent
Beneath Landscaped/Grass Areas	90%	As necessary to obtain density

During construction, we recommend that fill materials placed in the building area have a liquid limit of less than 45, and a plasticity index of less than 25. Whenever possible, highly plastic silt (MH) or clay (CH) fill soils should not be placed within the upper 4 feet of the final ground elevation. Soils which have a liquid limit greater than 45 and a plasticity index greater than 25 will typically require removal or blending with less plastic materials to result in lower Atterberg limits.

The soil horizons were categorized as per the Unified Soils Classification System (USCS) with additional notes regarding any soft, moist, or unsuitable soils. The presence and depth of subsurface water was estimated during excavation and measured after completion of each boring. The soil descriptions and classifications contained within the boring logs (Appendix B) were determined by visual observation of a Soils Engineer unless a laboratory number denotes the soil.

Graded Slopes

The site is observed to be mildly sloping down to the east. There is no major cut anticipated.

Fill Placement

Fill is not anticipated.

Foundation

Resistance to Lateral Loads

Lateral loads applied to foundations can be resisted by a combination of lateral bearing and base adhesion.

If the deflection resulting from the strain necessary to develop the passive pressure is within structural tolerance, the passive pressure and frictional resistance can be used in combination. Otherwise, additional passive pressure values could be provided based on tolerable deflection. The allowable values already incorporate a factor of safety and, as such, would be compared directly to the driving loads. If analytical approaches require the input of a ratio of available resisting forces and driving loads greater than unity, the ultimate values would be used.

Foundation Design (Conventional Shallow Foundations)

The proposed reconstruction may be adequately supported conventional shallow foundations.

a. All exterior wall foundations and interior bearing wall foundations shall extend not less than 18 inches and 16 inches, respectively, below undisturbed ground surface or finish grade (certified fill).

b. Exterior walls and interior bearing walls shall be supported on continuous foundations.

c. Exterior foundations shall be 12” with 18” minimum thickness reinforced with a minimum of two continuous horizontal reinforcing bars with at least two ½ inch diameter (# 4-bar) deformed reinforcing bars top and bottom and shall be placed 3 inches minimum concrete clearance.

e. Foundations for exterior walls and interior bearing walls shall be tied to the floor slabs by reinforcing bars (dowels) having a diameter of not less than ½ inch (# 4-bar) and spaced at intervals not exceeding 16 inches on center or as designed by a license designer. The reinforcing bars shall extend at least 40 bar diameters into the footings and the slab.

f. Pad footings shall be a minimum of 16”x 16” embedded 18” below native soil with 2-#4 deformed reinforcing bars each way or as designed by a license designer.

g. Concrete floor slabs-on-grade shall be cast over (Stegowrap 15 mil or equivalent) should be placed directly below the floor slab in order to reduce moisture condensation under the floor coverings

A minimum of 4 inch thick clean gravel base rock of ¾ inch diameter shall be used. The slab shall be at least 5 inches thick and shall be reinforced with #4-bar at 16 inches on center each way or as directed by the Project Structural Engineer.

h. The soil below an interior concrete slab shall be saturated with clean water to a depth of 12 inches prior to pouring the concrete.

i.The strength of concrete shall have an f'c = 2,500 psi minimum.

f. The allowable bearing capacity used should not exceed 2,500 psf for footings bearing on engineered fill. The allowable bearing capacity may be increased by one-third in the case of short duration loads, such as those induced by wind or seismic forces.

Drainage and Groundwater Considerations

The site should be graded to provide positive drainage to reduce storm water infiltration. Surface drainage should be planned to prevent ponding and to promote drainage of surface water away from the structure foundations, edges of pavements and sidewalks, toward suitable collection and discharge facilities. A minimum gradient of one percent for asphalt areas should be maintained. A three percent gradient should be maintained for landscaped areas immediately adjacent (within 10 feet) to the structure. In general, water should not be allowed to collect near the surface of the footing of the structures during or after construction. If water were allowed to accumulate next to the foundation, it would provide an available source of free water to the expansive soil underlying the foundation. Similarly, surface water drainage patterns or swales must not be altered so that runoff is allowed to collect next to the foundation.

Jobsite Safety

Neither the professional activities of GMD Engineers and sub consultants at a construction/project site, shall relieve the General Contractor of its obligations, duties and responsibilities including, but not limited to, construction means, methods, sequence, techniques or procedures necessary for performing, superintending and coordination the work in accordance with the contract documents and any health or safety precautions required by any regulatory agencies. GMD Engineers and its personnel have no authority to exercise any control over any construction contractor or its employees in connection with their work or any health or safety programs or procedures. The General Contractor shall be solely responsible for jobsite safety.

6.0 LIMITATIONS

Changed in the project design will render our recommendation invalid unless our staff reviews such changes and our specific recommendations are modified accordingly.

Our recommendations have been in accordance with the principles and practices generally employed by the soils engineering profession and engineering geology; and as such, this acknowledgement is in lieu of all other warranties, express or implied.

This report is being issued with the understanding that it is the responsibility of the Owner, or his representative, to ensure that the information and recommendations contained within our report are called to the attention of the Project Architect/ Engineers and incorporated into the plans, and that the necessary steps are being taken to ensure that the Contractors and Sub Contractors carry out such recommendations in the field.

Unanticipated soil and bedrock conditions are commonly encountered and cannot be fully evaluated by surface geologic investigations or exploratory borings, and frequently require that additional expenditures be made to attain proper development. Some contingency fund should be allotted to accommodate these possible extra costs.

We recommend the following:

1. We should be retained to provide observations and testing during removal of unsuitable soils, placement of select fill, preparation of subgrade, and construction observation of footing excavations.
2. We should be contacted with any questions that arise regarding application of our recommendations during construction, or if any soil conditions different from those described

in this report are encountered.

APPENDIX A

Unified Soil Classification System

Log of Test Boring

UNIFIED SOIL CLASSIFICATION SYSTEM – ASTM D2488 (Modified)

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN #200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN #4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels, gravel–sand mixtures, little or no fines
			GP	Poorly graded gravels or gravels–sand mixtures, little or no fines
		GRAVELS (MORE THAN 12% FINES)	GM	Silty gravels, gravel–sand–silt mixtures, non–plastic fines
			GC	Clayey gravels, gravel–sand–clay mixtures, plastic fines
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN #4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines
			SP	Poorly graded sands or gravelly sands, little or no fines
		SANDS (MORE THAN 12% FINES)	SM	Silty sands, sand–silt mixtures, non–plastic fines
			SC	Clayey sands, sand–clay mixtures, plastic fines
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN #200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 35%		ML	Inorganic silts and very fine clayey sand silt sands, with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly, sand, silty or lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS LIQUID LIMIT IS BETWEEN 35% AND 50%		MI	Inorganic silts, clayey silts and silty fine sands of intermediate plasticity
			CI	Inorganic clays, gravelly/sandy clays and silty clays of intermediate plasticity.
			OI	Organic clays and silty clays of intermediate plasticity.
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH	Organic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic clays
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils

BORING LOG EXPLANATION

LOGGED BY ______ DATE DRILLED _________ BORING DIAMETER _______ BORING NO. _____

Depth, ft.	Sample No. and Type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density, p.c.f.	Moisture % of Dry Wt.	MISC. LAB RESULTS
1			← Ground water elevation						
2	1–1		← Soil Sample Number	NOTE: All blows/foot are normalized to 2" outside diameter sampler size.					
3	L		← Soil Sample Size/Type L = 3" Outside Diameter M = 2.5" Outside Diameter T = 2" Outside Diameter ST= Shelby Tube BAG = Bag Sample						
4									
5									

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT
VERY LOOSE	0–4
LOOSE	4–10
MEDIUM DENSE	10–30
DENSE	30–50
VERY DENSE	OVER 50

CONSISTENCY

SILTS AND CLAYS	BLOWS/FOOT
VERY SOFT	0–2
SOFT	2–4
FIRM	4–8
STIFF	8–16
VERY STIFF	16–32
HARD	OVER 32

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Boring Log Explanation

Figure No.1

Project: Reconstruction of Single Family Dwellin	Project Number: 2019041	Client: Crossroads Christian Center	Boring No. B-1
Address, City, State 6235 Brookdale Dr Carmel CA		Drilling Contractor: CALIFORNIA GEOTECH	Drill Rig Type: B-24
Logged By: TM	Date — Started: 12/1/2019	Bit Type: 4-wing (solid head)carbide-tipped	Diameter: 4 inches
Drill Crew: CALIFORNIA GEOTECH	Date — Completed: 12/1/2019	Hammer Type:	
USA Ticket Number:	Date — Backfilled: Yes	Hammer Weight: 130 LBS	Hammer Drop: 0.762 m
	Groundwater Depth: NOT ENCOUNTERED	Elevation:	Total Depth of Boring: 20 ft

Depth (feet)	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	**Lithology** **Soil Group Name: modifier, color, moisture, density/consistency, grain size, other descriptors** **Rock Description: modifier color, hardness/degree of concentration, bedding and joint characteristics, solutions, void conditions.**	Dry Density (pcf)	Moisture Content (Optimum in %)	Field Penetrometer (tsf)
0					Organic soils & grass roots, dark brown clayey SAND, dense to medium dense, fine-grained, slight, dry			
	SPT	1a	12			102	14	2.3
					Dense			
10	SPT	1b	14			106	18	2.8
					Very dense			
	SPT	1c	18			110	18	3
					Sandstone-gray to orange staining, fracture, very dense			
20	SPT	1d	18		End of boring	118	24	3.5
					Groundwater not encountered at 15 feet during drilling.			

Standard Penetration Split Spoon Sampler (SPT)

California Sampler

Shelby Tube

CPP Sampler

Stabillized Ground water

Groundwater At time of Drilling

Bulk/ Bag Sample

GMD ENGINEERS

SOIL & FOUNDATION ENGINEERING

11 WEST LAUREL DRIVE SUITE 225 SALINAS CA 93906

(831) 840-4284

gmd.engr3@gmail.com

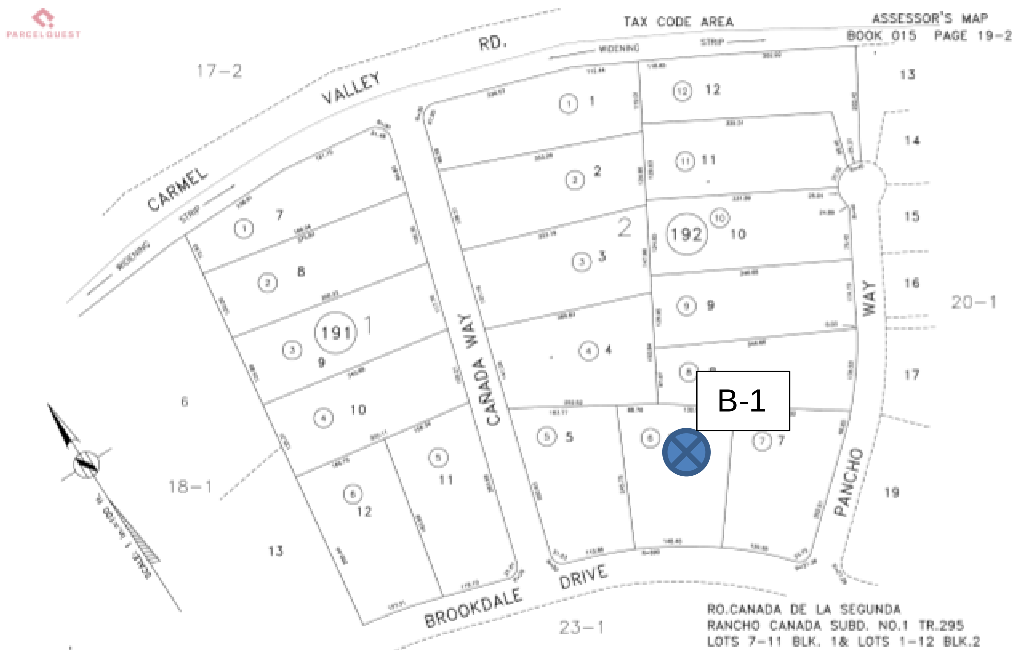
APN:015-192-006-000

PLATE NO: 1

APPENDIX "B"

LOCATION OF BORING

LOCATION PLAN



APPROXIMATE BORING LOCATION

6235 Brookdale Dr Carmel CA 93923

APN : 015-192-006-000

SITE MAP

6235 Brookdale Dr Carmel CA 93923

APN : 015-192-006-000

APPENDIX "C"

SEISMIC PARAMETERS

Results of Laboratory Soil Testing

Seismic Parameters

6235 Brookdale Dr Carmel CA 93923

6235 Brookdale Dr, Carmel-By-The-Sea, CA 93923, USA

Latitude, Longitude: 36.5377365, -121.8650448

Date	1/9/2020, 1:12:48 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.264	MCE_R ground motion. (for 0.2 second period)
S_1	0.471	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.264	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	0.842	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.549	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.604	Site modified peak ground acceleration
T_L	12	Long-period transition period in seconds
SsRT	1.264	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.374	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.965	Factored deterministic acceleration value. (0.2 second)
S1RT	0.471	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.513	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.716	Factored deterministic acceleration value. (1.0 second)

Type	Value	Description
PGAd	0.816	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.92	Mapped value of the risk coefficient at short periods
C_{R1}	0.918	Mapped value of the risk coefficient at a period of 1 s

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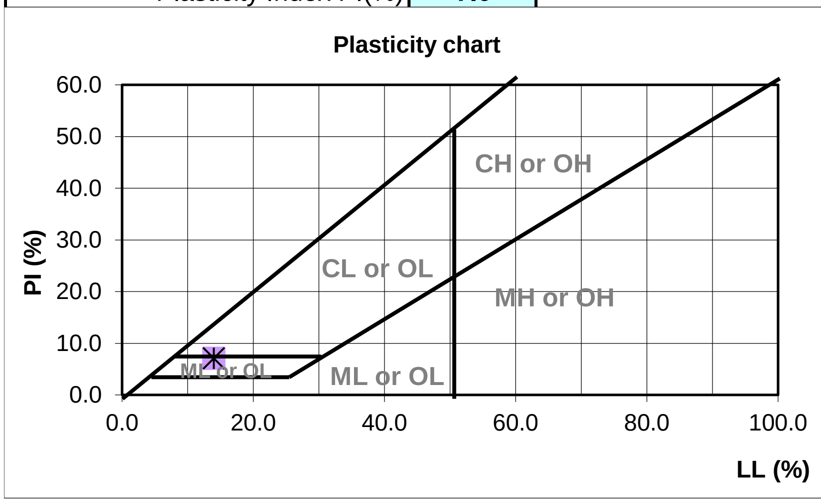
SOIL CLASSIFICATION ACCORDING ASTM

I. GRAIN SIZE PROPERTIES

Percent, gravel:	1%	0% – 55%
Percent, sand:	83%	
Percent, passing No 200:	45%	0% – 100%
		Gravel & Sand / Silt & Clay
Coefficient of uniformity Cu:		
Coefficient of curvature Cc:		

II. PLASTICITY OF FINES (PASSING SIEVE No.40)

Liquidity Limit LL (%)	14.0	☐ Organic with LLdry/LL<0.75
Plasticity Index PI(%)	7.0	



CLASSIFICATION RESULTS

Group symbol:

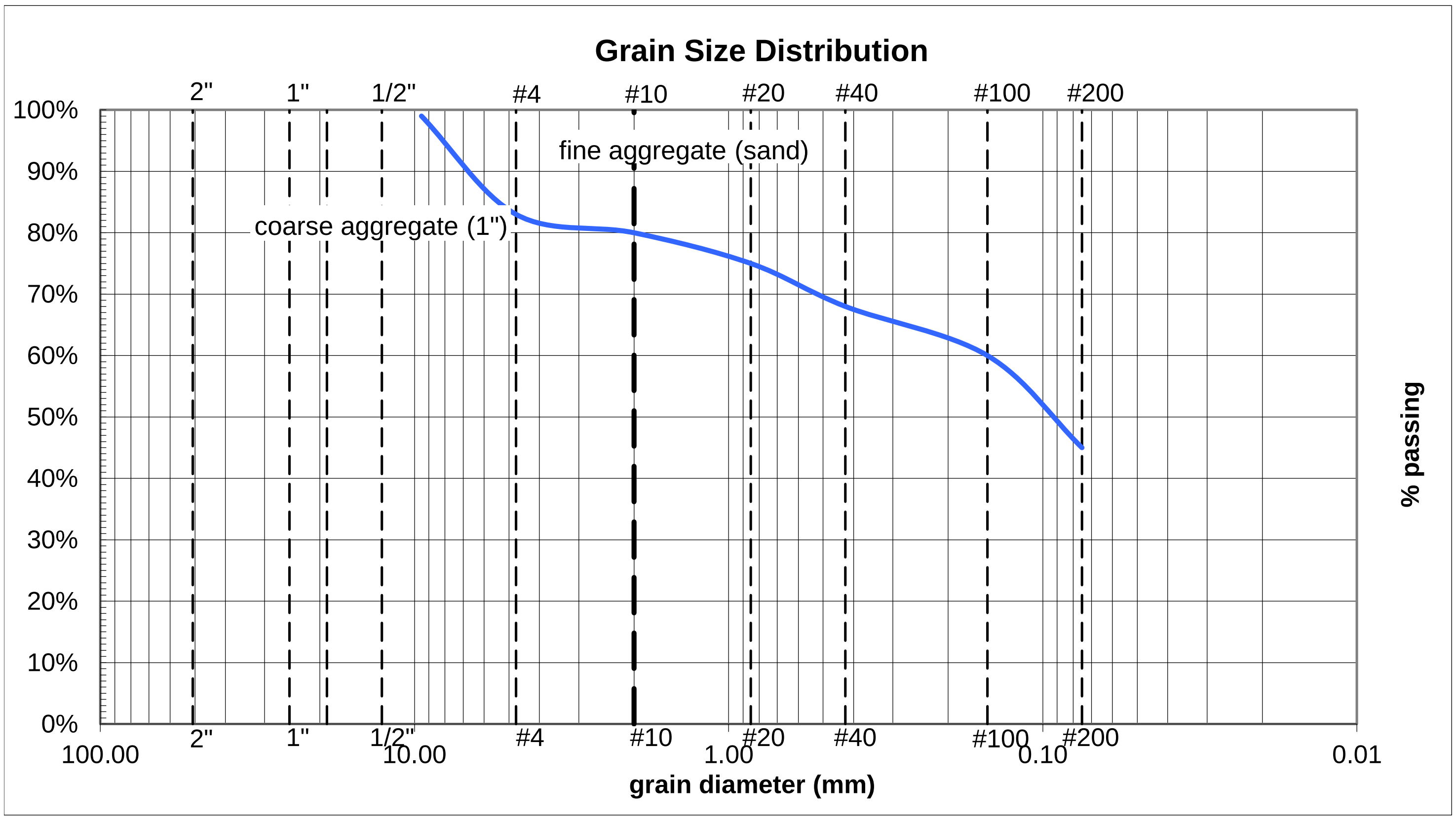
SC-SM

English designation as:

Silty, Clayey Sand

(It may be necessary to click on a random cell after changing input data in order to refresh the results)

GMD ENGINEERS SOIL & FOUNDATION ENGINEERING 11 WEST LAUREL DRIVE SUITE 225 SALINAS CA 93906 **(831) 840-4284** gmd.engr3@gmail.com	BORING #1 @ 5 FT APN:015-192-006-000 6235 Brookdale Dr Carmel CA 93923	GMD NO: 2019041 DATE: Dec 01, 2019 PLATE NO: 2



GMD NO: 201941
DATE: 12/01/2019
PLATE NO: A3

BORING 1 @ 5-6 FT

6235 Brookdale Dr
Carmel CA 93923
APN:015-192-006-000

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