# SOILS SOUTHWEST, INC.

SOILS, MATERIALS AND ENVIRONMENTAL ENGINEERING CONSULTANTS

897 VIA LATA, SUITE N • COLTON, CA 92324 • (909) 370-0474 • (909) 370-0481 • FAX (909) 370-3156


Report of Preliminary Soils and Foundation Evaluations
Proposed 11+ac. 77-lot Residential Development
35775 Iodine Springs Road
City of Wildomar
Riverside County, California
APN: 362-250-001 & 362-250-026

Project No. 15021-F

July 22, 2015

Prepared for:

Mr. Raffaele Suprano, CFO
% Marimina LLC.
245 Fischer Avenue, Suite A-8A
Costa Mesa, CA 92626


soilssouthwest@gmail.com
Established 1984

**SOILS SOUTHWEST, INC.**

SOILS, MATERIALS AND ENVIRONMENTAL ENGINEERING CONSULTANTS

897 VIA LATA, SUITE N • COLTON, CA 92324 • (909) 370-0474 • (909) 370-0481 • FAX (909) 370-3156

July 22, 2015

Project No. 15021-F

Nova Homes, Inc.
% Marimina LLC.
245 Fischer Avenue, Suite A-8A
Costa Mesa, CA 92626

Attention: Mr. Mr. Raffaele Suprano, CFO

Subject: Proposed 11+ac. 77-lot Residential Development
35775 Iodine Springs Road
City of Wildomar, Riverside County, California

Reference: Preliminary Conceptual Plan Prepared by Nova Homes, Inc.

Gentlemen:

Presented herewith are the Report of Preliminary Soils and Foundation Evaluations conducted for the site of the proposed 77-lot single family dwellings to be located at 35775 Iodine Springs Road, Wildomar, Riverside County, California. In absence of final grading and development plans, the recommendations supplied should be considered as "preliminary", subject to revisions following topographic, grading and development plan review.

The soils encountered consist, in general of, upper disturbed, dry and compressible, fine to medium coarse silty gravelly sands up to about 4 to 5 feet, overlying deposits dense, well cemented, coarse to very coarse, friable, gravelly sand of decomposed granitic origin. With the presence of minor pin-holes and traces of Caliche, laboratory testing dictates the natural deposits may be potentially susceptible to hydoconsolidation.

Based on review of the available published documents, it is our opinion that the subject site is not situated within an A-P Special Studies Zone, and the site should be considered non-susceptible to seismically induced soil liquefaction.

Based on the geotechnical evolutions completed as described herein, it is our opinion that the site should be considered suitable for the planned development, provided the recommendations presented are incorporated in final design and construction. The findings as described should be available to the project design professionals for their review and use. We offer no other warranty, express or implied.

Respectfully submitted,
Soils Southwest, Inc.

REGISTERED PROFESSIONAL ENGINEER
MOLOY K. GUPTA
NO. 31708 EXP. 12-31-16
CIVIL
STATE OF CALIFORNIA

Moloy Gupta, RCE 31708

John Flippin
Project Coordinator

dist/5-addressee

soilssouthwest@gmail.com
Established 1984

## Introduction

### 1.1 Purpose and Scope of Work

This report presents the preliminary results of Soils and Foundation Evaluations conducted for the site of the proposed 77-lot residential development to be located on the 11.7-acre parcel situated at 35775 Iodine Springs road, City of Wildomar, Riverside County, California. In absence of topographic, site grading and development plans, the recommendations supplied should be considered as "preliminary", subject to revision prior to actual grading and construction.

The soils/material descriptions included are based on visual observations made during test explorations conducted at this time, supplemented by laboratory testing completed as described.

The recommendations contained reflect our best estimate of the soils conditions encountered during field investigations conducted. It is not to be considered as a warranty of the soils for other areas, or for the depths beyond the explorations completed at this time.

The recommendations supplied should be considered valid and applicable when the following conditions, in minimum, are observed:

i. Pre-grade meeting with contractor, public agency and soils engineer,
ii. Excavated bottom inspections and verifications by soils engineer prior to backfill placement,
iii. Continuous observations and testing during site preparation and structural fill soils placement,
iv. Observation and inspection of footing trenching prior to steel and concrete placement,
v. Plumbing trench backfill placement prior to concrete slab-on-grade placement,
vi. On and off-site utility trench backfill testing and verifications, and
vii. Consultations as required during construction, or upon your request

### 1.2 Site Description

Primarily vacant and undeveloped, the uneven 11.7 acres site is bounded by a residential development currently underway on the north, by other undeveloped properties on the south, by the paved Iodine Springs Road on the east, and by George Street followed by other residential tract homes on the west. Overall vertical relief is currently unknown, but sheet-flow from incidental rainfall appear to flow towards the south. With the exception of weeds, minor scattered brush and isolated debris, presence of no other significant features pertinent to the planned development are noted.

### 1.3 Proposed Development and Grading

No grading plan, topographic map or detailed site improvement plans are available for review. However, based on the tentative development information supplied, it is understood that the subject site will be developed to include 77 detached single family dwellings, along with on and off-site street improvements. Use of conventional wood-frame and stucco construction are assumed with 2 kips and 15 klf for isolated pier and conventional wall loadings, respectively. Moderate site preparations and grading should be anticipated with the development currently planned.

## 1.4 Subsurface Investigation

Subsurface explorations consisted of 4 (four) geotechnical test explorations by using a 24-inch bucket backhoe advanced to a maximum 12 feet below grade. Prior to actual test explorations an underground utility clearance was made by the Underground Service Alert (USA) of Southern California to avoid possible subsurface life-line obstruction. The approximate test locations are shown on the attached Plate 1. Following necessary soil sampling and in-situ testing, the exploratory test trenches were backfilled with local soils using minimum compaction effort. During mass grading operations, areas of such should required additional compaction efforts using appropriate construction equipments.

During test excavations, representative bulk and undisturbed California ring samples were procured. Collected samples were subsequently sent to our laboratory for necessary geotechnical testing. It should be noted that with the presence of the friable, very dense, dry gravelly sandy soils retrievals of some of the undisturbed soils samplings were not complete.

## 1.5 Laboratory Testing

Representative bulk and undisturbed site soils samples were tested in our laboratory to aid in the soils classification and to evaluate relevant engineering properties pertaining to the project requirements. In general, the laboratory testing included the following:

- In-situ moisture contents and dry density (ASTM Standard D2216)
- Maximum dry density and optimum moisture content (ASTM Standard D1557)
- Direct Shear (ASTM Standard D3080)
- Soil Consolidation (ASTM Standard D2435)
- Soil gradation analysis (ASTM Standard D422)
- Soil Expansion index, EI (ASTM Standard D4829)

Description of the test results and test procedures used are provided in Appendix B.

o Based on the field investigation and laboratory testing, engineering analyses and evaluations were made on which to base our preliminary recommendations for design of foundations, slab-on-grade, paving and parking, site grading, utility trench backfill, site preparations and grading and monitoring during construction.

o Preparation of this report for initial use by the project design professionals. The recommendations supplied should be considered as 'tentative' and may require revision and/or upgrading following final development plan review.

# 2.0 Geotechnical Conditions

## 2.1 Site Soil Conditions

The soils encountered consist in general of upper disturbed, dry and compressible, fine to medium coarse gravelly sands up to about 4 feet, overlying natural deposits of light brown to brown, dense to very dense, well cemented, coarse to very coarse, friable, gravelly sand of decomposed granitic origin. With the presence of scattered pin-holes and Caliche, laboratory testing dictates the natural deposits may be potentially susceptible to consolidation (hydocollapsible) in contact with free water. Presence of no shallow depth bedrock or groundwater was detected.

The near surface loose and compressible soils existing as described are considered inadequate for directly supporting structural loadings without excessive differential settlements to load bearing footings and concrete slab-on-grade. When, however, graded in form of load bearing structural fill soils placement as recommended herein, the structural pads thus constructed for the dwellings planned should be adequate for load bearing support.

Laboratory shear tests conducted on the upper soils remolded to 90% indicate moderate shear strengths under increased moisture conditions. Results of the laboratory shear tests are provided in Plate B-1 of this report.

Slight soil compressibility is expected on similar remolded samples, thereby anticipating potential for "tolerable" settlements to footings and concrete slab-on-grade. Results of the laboratory determined soils consolidation potential is shown on Plate B-2 in Appendix B.

The near grade silty sandy alluviums exposed are considered "very low" in expansion characteristics requiring no special construction requirements when such are used overlain by concrete conventional slab on grade. Since soil matrix is expected to change following mass grading, it is our opinion that supplemental soil expansion potential verifications should be made to provide supplemental/revised recommendations, if warranted.

## 2.2 Subsurface Variations

During grading, buried irrigation, debris, organic and others may be encountered. In addition, variations in soil strata, their continuity and orientations may be expected. Due to the nature and depositional characteristics of the natural soils encountered, care should be exercised interpolating or extrapolating the subsurface soils conditions existing in between and beyond the test explorations conducted.

## 2.3 Excavatability

It is our opinion that the grading required for the project may be accomplished using conventional heavy-duty construction equipment. However, some difficulty may be expected during deep trenching due to soil caving. No blasting or jack-hammering, however, is anticipated.

## 2.4 Soil Corrosivity Potential

Since change in soil matrix is expected during site preparations and grading, no soil chemical analysis is initiated at this time to determine potentials for soils corrosivity to concrete and steel. Following mass grading completions evaluations on such will be made to determine, in minimum, pH, sulfate, chloride and resistivity. Results of such will be supplied, if and, when requested.

## 2.5 Groundwater

No shallow depth groundwater was encountered within the current maximum exploratory test depth of about at about 12 feet below existing grade. Based on review of the data available from the Water Library, historical shallowest groundwater level is estimated at about 7 feet below the current grade surface elevation of 1330.41.

While the groundwater described is expected not to affect the current planned development, it is our opinion, however, that provisions should be maintained so as to dispose of surface runoff away from the individual structural pads, once constructed.

Fluctuations in groundwater levels can occur due to seasonal variations in the amount of rainfall, runoff, altered natural drainage paths, and other factors not evident at the test explorations were completed. Consequently, the project civil engineer and grading contractor should establish a surface water runoff pattern so as to directed surface runoff away from the structural pads, once constructed.

## 3.0 Faulting And Seismicity

### 3.1 Faulting and Seismicity

Based on review of the information as published by the Department of Conservation, State of California, it is understood that the site is _not_ situated within an A-P Special Study Zone (*where a fault(s) runs through or adjacent to the development site*
However, considering the Southern California being in a seismically risky area susceptible to strong motion earthquake thereby causing structural damages, it is recommended that implementation of the current CBC seismic design parameters, along with the recommendations as described herein should be considered with the intention to “reduce” potentials for earthquake induced excessive structural distress, if any.

### 3.2 Direct or Primary Seismic Hazards

Surface ground rupture along with active fault zones and ground shaking represent primary or direct seismic hazards to structures. There are no known active or potentially active faults that pass through or towards the subject site, and the site is not situated within an AP Special Studies Zone. According to the 2013 current CBC, the site is considered within Seismic Zone 4. As a result, it is likely that during the life expectancy of the structure built, moderate to severe ground shaking may have potential for adverse effects on the site.

### 3.3 Induced or Secondary Seismic Hazards

In addition to ground shaking, effects of seismic activity may include surface rupture, flooding, land-sliding, lateral spreading, settlements and subsidence. Potential effects of such are as described below.

### 3.4 Liquefaction

Liquefaction is caused by build-up of excess hydrostatic pressures in saturated cohesionless soils due to cyclic stress generated by ground shaking. The significant factors on which liquefaction potential of a soil deposit depends, among others, include soil type, relative soil density, intensity of earthquake, duration of ground shaking, and depth of ground water, among others.

Although no site-specific site soils liquefaction susceptibility potential is currently included, it is our opinion, however, that considering the presence of very dense gravelly sandy soils or granitic bedrock at depth as described in Test Boring Logs, the site soils should be considered non-susceptibility to seismically induced liquefaction. Further evaluations on such may be initiated, if and, on request.

### 3.5 Shallow Ground Rupture

The site is not situated within an AP Special Studies Zone. Based on review of existing geologic information, no major fault is noted to cross through or extends towards the site. The potential for surface rupture resulting from nearby fault movement is not known for certainty, but is considered "remote" due to the distance of the site with respect to the nearby earthquake fault identified as described.

### 3.6 Flooding

Flooding hazards include tsunamis (seismic sea waves), Seiches, and failure of manmade reservoirs, tanks and aqueducts. The potential for these hazards is considered "remote" considering the inland site location and absence of any nearby any nearby bodies of water or storage reservoir.

### 3.7 Landslides

Seismically induced landslides and other slope failures are common occurrences during or soon after an earthquake. Considering that the subject site and its adjacent being relatively flat, it is our opinion that potential for seismically induced landslides should be considered as "remote".

### 3.8 Lateral Spreading

Seismically induced lateral spreading involves lateral movement of soils due to ground shaking. Lateral spreading is demonstrated by near vertical cracks with predominantly horizontal movement of the soil mass involved. The topography of the site being near level, it is our opinion that the potential for seismically induced lateral spreading should be considered as "remote".

### 3.9 Seismically Induced Settlement and Subsidence

The site is situated at about 1.56 km from the Elsinore GL+T Fault capable of generating an earthquake magnitude M=7.2 and Peak Ground Acceleration, PGA, of 0.532g. Considering the proximity of the earthquake fault described, it is our opinion that potential for some settlement may be anticipated during life-time use of the structures once built. Although no evaluations are currently made to estimate extents of potential ground settlement, it is our opinion that adverse effects of such on the structures built may be minimized by using appropriate structural deign by using the guidelines presented in the current CBC and that of the requirements of the local public agency as dictates. If and when requested, supplemental settlement analyses may be programmed following additional field explorations and field testing (SPT) using a drillrig.

### 3.10 Seismic Design Parameters

The design spectrum was developed based on the 2013 CBC. Site Coordinate, of 33.300480°N, -117.238968°W were used to establish the seismic parameters presented below

### 3.11 Seismic Design Coefficients as per 2013 CBC

The site is situated at about 1.56 km from the Elsinore GL+T Fault. For foundation and structural design, based on the current CBC, the following seismic design parameters are suggested.

Recommended values are based upon USGS Design Maps Summary and Detailed Reports website for Mapped Acceleration Parameters, USGS 2008 National Seismic Hazard Maps-Fault Parameters, and the California Geologic Survey: Probabilistic Seismic Hazards Mapping and supplemental seismic parameters are provided in Appendix C of this report.

The following presents the seismic design parameters as based on available publications as currently published by the California Geological Survey and 2013 CBC

**TABLE 3.11A.1 Seismic Design Parameters**

| CBC Chapter 16 Paragraph/Table | 2013 ASCE 7 Standard Seismic Design Parameters | Recommended Values |
|---|---|---|
| 1613A.5.2 | Site Class | C |
| 1613.5.1 | The mapped spectral accelerations at short period | $S_s$ |
| 1613.5.1 | The mapped spectral accelerations at 1.0-second period | $S_1$ |
| 1613A5.3(1) | Site Class B / Seismic Coefficient, $S_s$ | 2.279 g |
| 1613A5.3(2) | Site Class B / Seismic Coefficient, $S_1$ | 0.921 g |
| 1613A5.3(1) | Site Class D / Seismic Coefficient, $F_a$ | 1.000 g |
| 1613A5.3(2) | Site Class D / Seismic Coefficient, $F_v$ | 1.500 g |
| 16A-37 Equation | Spectral Response Accelerations, $S_{Ms} = F_a S_s$ | 2.279 g |
| 16A-38 Equation | Spectral Response Accelerations, $S_{M1} = F_v S_1$ | 1.198g |
| 16A-39 Equation | Design Spectral Response Accelerations, $S_{Ds} = 2/3 \times S_{Ms}$ | 1.519 g |
| 16A-40 Equation | Design Spectral Response Accelerations, $S_{D1} = 2/3 \times S_{Ms}$ | 0.799 g |

**TABLE 3.11A.2 Seismic Source Type**

Peak Horizontal Ground Acceleration (PHGA) is based on an earthquake having a 10 percent probability of exceedance in a 50 year period.

| Seismic Source Type / Appendix C | |
|---|---|
| Nearest Maximum Fault Magnitude | M>\=7.2 |
| Peak Horizontal Ground Acceleration | 0.450g - 0.532g |

In design, vertical acceleration may be assumed to about 1/2 of the estimated horizontal ground accelerations described.

It should be noted that lateral force requirement in design by structural engineer should be intended to resist total structural collapse due to the described PHGA or greater. However, during life time use of the structure built, it is our opinion that some structural damage may be anticipated thereby requiring some structural repairs. Adequate structural design and implementation of the current CBC design requirements should be strictly observed. To minimize potentials for rupture during an earthquake, use of flexible lifelines for gas, water, electricity and others, are strongly suggested.

# 4.0 Recommendations

## 4.1 General Evaluations

Based on field explorations, laboratory testing and subsequent engineering analysis, the following conclusions and recommendations are presented for the site under study:

(I) From geotechnical viewpoint, the site is considered grossly stable for the proposed development, provided the recommendations supplied herein are incorporated in design and construction. Foundation design should reflect considerations of the seismically induced PHGA as described.

(II) Because of the dry, disturbed, and compressible nature of the upper soils as encountered, it is our opinion that for structural support, the near surface soils should be reworked in form of subexcavations, followed by scarification, moisturization and replacement of the excavated soils to planned grades compacted to minimum 90%. In event new fill soils are required over the current grade surface such should be placed on the prepared subgrades as described.

(III) The subexcavation depth described in the following section should be considered as "minimum". During grading, localized deeper subexcavations may be required within areas underlain by buried debris, utilities and others. It will be the responsibility of the grading contractor to inform the project soils engineer of the presence of such debris or utilities such as septic tank etc.

(IV) In order to minimize potential differential settlements, it is recommended that structural footings should be established exclusively into engineered fills of local soils compacted to the minimum requirements as described in this repor. Construction of footings and slabs straddling over cut/fill transition, shall be avoided.

(V) Structural design consideration should include probability for moderate to high peak ground acceleration from relatively active nearby earthquake faults. Implementing, however, it is our opinion that the adverse effects of ground shaking may be minimized using the design guidelines as described in the current CBC.

(VI) Although no groundwater was encountered within the depths explored, provisions should be maintained during construction to divert incidental rainfall away from the structural pads constructed.

(VII) It is our opinion that, if site preparations and grading are performed as recommended and as per the generally accepted construction practices and current CBC, the proposed development will not adversely affect the stability of the site, or it's adjacent.

### 4.1.1 Site Preparations (general)

The site preparations within the planned structural pads and beyond should include complete removals of the surficial weed, and scattered debris, followed by subexcavations to the approximate depths as described in his report. Site preparations should also include stock-piling of the subexcavated soils and moisturization of the surface exposed to about 3% to 5% over optimum moisture content, followed by its recompaction prior to the approved stockpile soil replacement as engineered fills compacted to desired pad grades. During grading, the local excavated soils stockpiled should be spread in 6 to 8-inch thick compacted lifts to pad finish grades compacted to minimum 90%.

### 4.1.2 Preparations for Structural Pads (Specific)

In absence of site-specific grading, topographic or development plan the following general recommendations are provided for "estimation" purposes, subject to revision prior to actual grading and construction.

Considering the upper existing disturbed or loose dry, low density and compressible soils encountered as described, it is recommended that no structural footings and/or paving shall be constructed bearing directly on the near surface soils currently existing. Additionally, "transition" conditions should be allowed underneath footings and slabs straddling over cut and fill subgrades and no rocks larger that 6-inch are diameter should be allowed directly underneath load bearing foundations. Overnight 'flooding' prior to actual site preparations and grading, may be considered..

In absence of precise grading/development plan, it is assumed that the subject development will be located either on (A) near existing grade, or (B) on pads constructed by minor fill soils placement over the current grade surface, or (C) by minor cuts to the grades currently existing.

For the development planned, it is assumed that wood frame and stucco construction will be used using conventional spread footings measuring approximately 12"x12" and 15"x18' for 1-story and 2-story structures, respectively. Actual foundation dimensions should be supplied by the project structural engineer based on static vertical loading and soil bearing capacity, along with the requirements of the current CBC and the seismically induced ground accelerations as described earlier.

The planned site preparations and grading required for structural pads proposed should include (i) subexcavations of the existing soils, (ii) followed by replacement of the same in thin lifts compacted to minimum 90%, or better. For the pads planned at near or on the existing grade surface, site preparations should include subexcavations measuring vertically either to:

A> For the planned pads requiring new structural fill soils placement on existing grades, site preparations prior to new soils placement should include subexcavations of the upper dry and loose soils to (I) *a minimum 5 feet* below the present grade surface, or (II) to the depth of the underlying moist and dense gravelly sandy natural soils, (iii) or to the depth as required to maintain a *24-inch thick compacted fill mat blanket below the planned footing bottoms,* whichever is greater.

B> For the pads requiring "cuts" to the present grades, the site preparations, following such planned cuts, should include further subexcavations of the excavated bottoms to the sufficient depth so as to maintain a minimum 18-inch thick compacted fill mat blanket below the planned footing bottoms, or to the depth as recommended by soils engineer during grading.

C> For cut/fill transition areas, it is recommended that following cuts to planned grade, the cut portions of the pads should be further subexcavated to sufficient depth so as to maintain an overall minimum compacted fill mat blanket as described below:

### 4.1.3 Cut/Fill Transition Pad Preparations (General)

Cut/fill transitions within structural pads should be avoided to minimize potentials for differential settlements to footings and concrete slab-on-grade where required fill depth exceeds planned footing depth. Within areas of cut-fill transition, it is suggested that following necessary subexavations within cut areas, the entire structural pad should be established on uniform bearing compacted fills with the grading guidelines as described below.

*Pad Preparation Guideline for Cut/Fill Transition Areas*

| Fill Depth Required for Finish Grade (within low-lying areas) | Overexcavation Depth below Finish Grade (within cut areas) |
|---|---|
| Up to 5 feet | Equal Depth |
| 5 to 10 feet | 5 feet |
| Greater than 10 feet | One-half the maximum thickness of fill placed on the "fill" portion (20 feet maximum) |

Cut portions should be over-excavated beyond the structural perimeter lines a horizontal distance equal to the depth of over excavation or to a minimum distance of 5 feet, whichever is greater. Actual subexcavation depths, however, should be determined by soils engineer during grading.

The subexcavation depths described should be considered "approximate". Actual subexcavation depths should be determined in field by the project soils engineer during grading. The site grading procedures described should, in minimum, encompass the planned building footprint areas and five (5) feet beyond. Imported fill soils, if required, should be approved by soils engineer prior to their use.

Supplemental general grading recommendations are provided in Section 5 of this report.

### 4.1.4 Structural Fill Soils Requirements

(i) Non-expansive in nature, the on-site soils free of organic, debris and rocks larger than 6-inch in diameter, should be considered suitable for re-use as structural backfills.

(ii) Following mass-grading completion, representative site soils sampled from graded fills expected in contact with footings and utilities should be laboratory tested to verify presence of Sulfate, pH, chloride and Resistivity. Based on the chemical test results, supplemental design recommendations will be supplied prior to concrete pour. Such chemical testing will be programmed, if and when requested by the addressee.

## 4.2 Recommendations for Load Bearing Spread Foundation Design

For the development proposed, use of light-loaded conventional wood frame construction with concrete slab-on-grade and load bearing spread foundations, are anticipated. Structural loadings are assumed within not to exceed 2 klf and 30 kips for continuous wall and isolated pier foundations, respectfully. The one to two-story structures are expected to be supported by continuous wall and/or isolated spread footings founded exclusively into engineered fills of local gravelly sandy soils or its equivalent compacted to minimum 90%.

For design, an allowable soil vertical bearing capacity of 1800 psf may be considered. If normal code requirements are applied, the above capacities may further be increased by an additional 1/3 for short duration of loading which includes the effect of wind and seismic forces.

To minimize potential differential settlements, use of footings straddling over cut/fill transition, shall be avoided. Considering dry gravelly nature, it is recommended that the excavated footing trenches should be sufficiently "moistened" immediately prior to concrete placement.

Considering the presence of the nearby earthquake faults as described earlier, it is recommended that, unless specified otherwise by the project structural engineer, from geotechnical view point, conventional footings may be sized to a minimum 12"x12" and 15"x18" for one and two-story construction, respectively. The footing depths described should be measured vertically from the *lowest adjacent outside grade, and NOT from the finished pad grade or finished floor surface.* Footing depths and dimensions shall be verified by soils engineer prior to footing-forming, rebar and concrete placement.

It will be the contractor's responsibly to arrange footing verification by soils engineer.

Structural design should conform to the current CBC Seismic Design, including the PHGA requirements as described earlier in Section 3.6 of this report.

From Static Loading conditions, footing reinforcements consisting of 2-#4 rebar placed near the top and 2-#4 rebar near bottom of continuous footings, are recommended. Additional reinforcements, as specified by project structural engineer, should be incorporated during construction.

The settlements of properly designed and constructed foundations supported on engineered fill, comprising of site soils or its equivalent or better, and carrying maximum anticipated vertical loadings, are expected to be within tolerable limits. Estimated total and differential settlements are about 1 and 3/4-inch, respectively. Considering the sandy nature of the soils as encountered, it is our opinion that most of the elastic deformations should be expected immediately during construction.

### 4.3 Concrete Slab-on-Grade

No concrete slabs, sidewalks and flatworks should be placed bearing directly on the surface soils existing. The prepared subgrades to receive footings should be adequate for concrete slab-on-grade placement. Considering the close proximity of earthquake faults, 4-inch thick concrete slabs reinforced with #3 rebar at 18-inch o/c is recommended or as designed by the structural engineer based upon structural loading requirements for the seismic design parameters and for the horizontal peak ground acceleration (PGA) as described in this report. Additionally, concrete slabs must maintain positive contact with footings by use of dowels, or similar means as designed by the project structural engineer.

For driveways, it is our opinion that concrete slabs should be 5-inch thick, placed over local gravelly sandy soils compacted to at least 95%. Driveway slab reinforcing and construction and expansion joints etc. should be incorporated if required by the project structural engineer.

Within moisture sensitive areas, concrete slabs should be underlain by 2-inch of compacted clean sand, followed by 6-mil thick vapor barrier such as commercially available StegoWrap,

Visqueen or other approved coverings, underlain by an additional 2-inch of approved sand covering. The gravelly sands used underneath concrete slabs should have a Sand Equivalent, SE, of 30 or greater.

Subgrades to receive concrete should be "'dampened" as would be expected in any such concrete placement. Use of low-slump concrete is recommended. In addition, it is recommended that utility trenches underlying concrete slabs and driveways should be thoroughly backfilled with gravelly sandy soils mechanically compacted to minimum 90%. Slab subgrade verification by soils engineer is required prior to vapor barrier placement. No water jetting should be allowed in an effort to compact utility trench backfills.

### 4.3.1 Concrete Curing

In order to minimize potential for excessive concrete shrinkage or cracking, concrete slabs shall be 'cured' by using water for at least 7 days or as determined by the structural engineer prior to structural load placement.

## 4.4 Resistance to Lateral Loads

Resistance to lateral loads can be restrained by friction acting at the base of foundation and by passive earth pressure. A coefficient of friction of 0.3 may be assumed with normal dead load forces for footing established on compacted fill.

An allowable passive lateral earth resistance of 250 pounds per square foot per foot of depth may be assumed for the sides of foundations poured against compacted fill. The maximum lateral passive earth pressure is recommended not to exceed 2500 pounds per square foot.

For design, lateral pressures from local soils when used as level backfill may be estimated from the following equivalent fluid density:

*Lateral Earth Pressures*

| CONDITIONS | EQUIVALENT FLUID WEIGHT(pcf) | |
|---|---|---|
| | Level Backfill | 2:1 Backfill Sloping Upwards |
| Active | 35 | 55 |
| At Rest | 60 | 73 |
| Passive | 250 | - |

## 4.5 Swimming Pool (If planned)

For adequate support, it is recommended that the swimming pool shell should be founded exclusively on underlying competent natural subgrade. For design, the following criteria may be considered:

1. Swimming pool full, with no passive resistance;
2. Swimming pool empty, with lateral active pressures from surrounding soils;
3. Swimming pool full, with supported soil surrounding.

With soil vertical bearing capacity of 1800 psf, for design, lateral active pressures and passive resistance in form of "equivalent fluid density" from horizontal backfill, may be considered from the Table 2.0 described.

## 4.6 Shrinkage and Subsidence

It is our opinion that during grading the upper soils may be subjected to a volume change. Assuming a 90% relative compaction for structural fills and assuming an overexcavation and re-compaction depth as described, such volume change due to shrinkage may be on the order of 10 to 15 percent. Further volume change may be expected due to supplemental shrinkage during preparation of subgrade soils. For estimation purpose, such may be approximated to about 2-inch when conventional construction equipments are used.

## 4.7 Construction Consideration

### 4.7.1 Unsupported Excavation

Gravelly sandy site soils encountered are considered highly susceptible to caving. Temporary excavations up to 5 feet may be made without rigorous lateral supports. Excavated surface should be "'wetted" during construction in order to minimize potential surface soil raveling. No surcharge loading should be allowed within an imaginary 1:1 line drawn upward from toe of temporary excavations.

### 4.7.2 Supported Excavations

If vertical excavations exceeding 5 feet in depths become warranted, such should be achieved using shoring to support sidewalls.

## 4.8 Soil Caving

Considering dry gravelly in nature, the site soils are considered "highly" susceptible to caving. Temporary excavations in excess of 5 feet should at a slope 2 to 1 (h:v), or flatter, and as per the construction guidelines provided by the Cal-Osha.

## 4.9 Structural Pavement Thickness (Tentative)

### Flexible Asphalt Paving

Anticipating change in soil-matrix following mass grading operation, no actual soil R-value determination is made at this time. Based on estimated Traffic Index (TI) and on assumed soil R-value of 65, the following paving sections are supplied for estimation purposes. Prior to actual paving soil R-value should be determined on samples procured from the planned street grade based on which actual paving sections should be determined to be used. For estimation purposes, the following tentative paving sections may be considered.

**Table 3.0 - Preliminary Pavement Design**

| Preliminary Asphalt Concrete (AC) Pavement Design (Off-Site) | |
|---|---|
| Assumed Traffic Index | 7.0 |
| R-value (assumed) | 65 |
| AC Thickness (inches) | 4.0* |
| AB Thickness (inches) | 6.5* |

*Notes: AC - Asphaltic Concrete*
*AB - Aggregate CLASS II "clean" Base*
** Should meet or exceed City of Rancho Cucamonga Minimum Thickness Requirements*

For a.c over base a minimum upper 12-inch of the subgrade soils should be compacted to 90%.

Base material used should conform to Caltrans Class II specification compacted to minimum 95%. Paving sections supplied; should be verified by the City for their minimum pavement thickness requirements.

### 4.10 Private Concrete Flatwork/Driveways

Concrete flatworks (such as walkways and driveways) have potential for cracking due to fluctuations in soil volume in relationship to moisture content changes. In order to prevent excessive cracking or lifting, concrete paving should meet the minimum guidelines as shown in the table below. It is our opinion that when designed and adequately constructed, the following guidelines will help “reduce” potential for irregular cracking or lifting, but will not eliminate all concrete distress.

| | ***Private Sidewalks*** | ***Private Drives*** | ***Patios/Entryways*** | ***City Sidewalk/Curb and Gutters*** |
|---|---|---|---|---|
| ***Minimum Thickness (in.)*** | 4 (nominal) | 4 (full) | 4 (full) | City/Agency Standard |
| ***Pressoaking (+/-2% Optimum)*** | 12 inches | 12 inches | 12 inches | City/Agency Standard |
| ***Reinforcement*** | — | No. 3 at 24 inches on centers | No. 3 at 24 inches on centers | City/Agency Standard |
| ***Thickness Edge*** | — | 8" x 8" | 8" x 8 " | City/Agency Standard |
| ***Crack Control*** | Saw cut or deep open tool joint to a minimum of 1/3 of concrete thickness | Saw cut or deep open tool joint to a minimum of 1/3 of concrete thickness | Saw cut or deep open tool joint to a minimum of 1/3 of concrete thickness | City/Agency Standard |
| ***Maximum Joint Spacing*** | 5 feet | 10 feet or quarter cut whichever is closer | 6 feet | City/Agency Standard |

No concrete slabs, sidewalks and flatworks should be placed bearing directly on the surface soils currently existing. The prepared subgrades to receive footings should be adequate for concrete slab-on-grade placement. The maximum density of the base material should be more than its supporting subgrade material.

Unless otherwise specified in this report or by the local public agency, the following guidelines may be considered in subgrade preparations for the paving described:

| Type of Compaction | Minimum Required Compaction (%) |
|---|---|
| Sidewalks,Patios, Paths,Breezeways | 90 |
| Concrete Slab | 90 |
| Driveways, Parking, Ramps | 95 |
| Street/Driveway Subgrade ***with*** base | 90 |
| Street/Drive Subgrade ***without*** base | 95 |
| Curb and Gutter/V-Gutter ***with*** base | 90 |
| Curb and Gutter/V-Gutter ***Without*** base | 95 |
| Base and Asphalt | 95 |

Driveway slab reinforcing and construction and expansion joints etc. should be incorporated if required by the project structural engineer.

Within moisture sensitive areas, concrete slabs should be underlain by 2-inch of compacted clean sand, followed by 6-mil thick Visqueen. The gravelly sands used should have a Sand Equivalent, SE, of 30 or greater.

Subgrades to receive concrete should be "pre-moistened" as would be expected in any such concrete placement. Use of low-slump concrete is recommended. In addition, it is recommended that utility trenches underlying concrete slabs and driveways should be thoroughly backfilled with gravelly sandy soils mechanically compacted to minimum 90% (+2 feet below final grade) and 95% (0-2 feet below final grade) immediately prior to concrete pour.

## 4.11 Boundary Wall/Retaining Wall

It is our opinion that retaining structure, if planned, should be designed based on following parameters:

| Slope of Retained Material (H:V) | Equivalent Fluid Density, pcf Clean Sand | Local Soil |
|---|---|---|
| level | 30 | 35 |
| 2:1 | 42 | 55 |

Walls adjacent to traffic areas should be designed to resist a uniform lateral pressure of 100 pounds per square foot, which is a result of an assumed 300 pounds per square foot surcharge behind the walls due to normal traffic. If the traffic is kept back ten feet from the wall, the traffic surcharge may be neglected. The design parameters do not include any hydrostatic pressure build-up. Consequently, installation of "'french-drain" behind retaining walls is recommended to minimize water pressure build-up behind retaining walls. Use of impervious material is preferred within upper 18 inches of the backfill placed.

Backfill behind retaining wall should be compacted to a minimum 90 percent relative laboratory Maximum Dry Density as determined by the ASTM D15571 test method. Flooding and/or jetting behind wall should not be permitted. Local sandy soils may be used as backfill. Supplemental geotechnical specifications on such will be supplied following construction details review.

### 4.12 Utility Trench Backfill

Utility trench backfill within the structural pad and beyond should be placed in accordance with the following recommendations:

- o Trench backfill should be placed in 6 to 8-inch thin lifts mechanically compacted to 90 percent or better of the laboratory maximum dry density for the soils used. Jetting is not recommended within utility trench backfill. Within streets, upper 2 feet of the trench backfill should be compacted to 95% or better.

- o Exterior trenches along a foundation or a toe of a slope and extending below a 1:1 imaginary line projected from the outside bottom edge of the footing or toe of the slope should be compacted to 90 percent of the Maximum Dry Density for the soils used during backfill. Excavations should conform to the requirements of Cal-Osha

### 4.13 Pre-Construction Meeting

It is recommended that no clearing of the site or any grading operation be performed without the presence of a representative of this office. An on-site pre-grading meeting should be arranged between the soils engineer and the grading contractor prior to any construction.

### 4.14 Seasonal Limitations

No fill shall be placed, spread or rolled during unfavorable weather conditions. Where the work is interrupted by heavy rains, fill operations shall not be resumed until moisture conditions are considered favorable by the soils engineer.

### 4.15 Planters

In order to minimize potential differential settlement to foundations, use of planters requiring heavy irrigation *should be restricted from using adjacent to structural footings*. In event such becomes unavoidable, planter boxes with sealed bottoms, should be considered.

### 4.16 Landscape Maintenance

Only the amount of irrigation necessary to sustain plant life should be provided. Pad drainage should be directed towards streets and to other approved areas away from foundations. Slope areas should be planted with draught resistant vegetation. Over watering landscape areas could adversely affect the proposed site development during its life-time use.

## 4.17 Observations and Testing During Construction

Recommendations provided are based on the assumption that structural footings and slab-on-grade be established exclusively into compacted fills. Excavated footings should be inspected, verified and certified by soils engineer prior to steel and concrete placement to ensure their sufficient embedment and proper bearing as recommended. Structural backfills discussed should be placed under direct observations and testing by this facility. Excess soils generated from footing excavations should be removed from pad areas and such should not be allowed on subgrades underlying concrete slab.

## 4.18 Plan Review

No precise grading or development plans are prepared and none such is available for review. Prior to actual mass grading, grading and foundation plans should be available for review to ensure applicability of the assumptions made in preparing this report. If during construction, conditions are observed different from those as presented, revised and/or supplemental recommendations will be required.

## 5.0 Earth Work Guidelines

The project area is currently underlain with dry loose silty sandy alluviums with minor rocks. Prior to grading commencement, it is suggested that any and all debris and loose stockpiles etc., should be cleared and disposed off of-site to the satisfaction of soils engineer. In general, site preparations and grading for the project should include, in minimum the following:

### Structural Backfill:

Local soils free of organic, debris and rocks larger than 6-inch in overall diameter should be considered suitable for reuse as structural backfill. Loose soils, formwork and debris should be removed prior to backfilling retaining walls. Local soils backfill should be placed and compacted in accordance with the recommended specifications provided below. Where space limitations do not allow conventional backfilling operations, special backfill materials and procedures may be required. Pea gravel or other select backfill can be used within limited space areas. Additional recommendations on such will be provided during construction.

### Site Drainage:

Adequate positive drainage should be provided maintained away from structural pad in order to prevent water from ponding and to reduce potential percolation into backfill. A desirable slope for surface drainage is 2 percent in landscape areas and 1 percent in paved areas. Planters and landscaped areas adjacent to building perimeter should be adequately designed to minimize water filtration into subsoils. Considerations should be given to the use of closed planter bottoms, concrete slabs and perimeter subdrains where applicable.

### Utility Trenches:

Buried utility conduits should be bedded and backfilled around the conduit in accordance with the project specifications. Where conduit underlies concrete slab-on-grade and pavement, the remaining trench backfill above the pipe should be placed and compacted to at least 90%.

### General Grading Recommendations:

Recommended general specifications for surface preparation to receive fill and compaction for structural and utility trench backfill and others are presented below.

1. Areas to be graded, backfilled or paved, shall be grubbed, stripped and cleaned of all buried and undetected debris, structures, concrete, vegetation and other deleterious materials prior to grading.

2. Where compacted fill is to provide vertical support for foundations, all loose, soft and other incompetent soils should be removed to full depth as approved by soils engineer, or at least up to the depth as previously described in this report. The areas of such removal should extend at least 5 feet beyond the perimeter of exterior foundation limit or to the extent as approved by soils engineer during grading.

3. The recommended compaction for fill to support foundations and slab-on-grade is 90% of soil's Maximum Dry Density at or near Optimum Moisture Content. To minimize potential differential settlements to foundations and slabs straddling over cut and fill transition, cut portions following cut, should be further over-excavated and such be replaced as engineered fill compacted to at least 90% of the soil's Maximum Dry Density as described in this report.

Utility trenches planned within building pad areas and beyond should be backfilled with granular material and such should be mechanically compacted to at least 90% of the maximum density for the material used. No water jetting shall be allowed for compaction in lieu of mechanical compaction.

4. Compaction for structural fills shall be determined relative to the maximum dry density as determined by ASTM D1557 compaction methods. All in-situ field density of compacted fill shall be determined by the ASTM D1556 standard methods or by other approved procedures.

5. All new imported soils, if required, shall be clean, granular, non-expansive material requiring prior approval by soils engineer.

6. During grading, fill soils shall be placed as thin layers, thickness of which following compaction shall not exceed six inches.

7. In accordance with the CBC; rock sizes greater than 12 inches (305 mm) and up to 24 inches (610 mm) in maximum dimension shall be three feet (914 mm) or more below grade, measured vertically. Rock sizes greater than 24 inches (610 mm) in maximum dimension shall be 10 feet (3048 mm) or more below grade, measured vertically.

8. No jetting and/or water tampering be considered for backfill compaction for utility trenches without prior approval of the soils engineer. For such backfill, hand tampering with fill layers of 8 to 12 inches in thickness or as approved by the soils engineer is recommended.

9. Any and all utility trenches at depth as well as cesspool and abandoned septic tank within building pad area and beyond, should either be completely excavated and removed from the site, or should be backfilled with gravel, slurry or by other material, as approved by soils engineer.

10. Any and all import soils if required during grading should be equivalent to the site soils or better. The soils engineer prior to their use should approve such.

11. Any and all grading required for pavement, side-walk or other facilities to be used by general public, should be constructed under direct observation of soils engineer or as required by the local public agencies.

12. A site meeting should be held between grading contractor and soils engineer prior to actual construction. Two days of prior notice will be required for such meeting.

## 6.0 Closure

The conclusions and recommendations presented are based on the findings and observations made at the time of subsurface test explorations. However, the recommendations supplied should be considered "preliminary" since they are based upon soil samples only. Supplemental investigations and additional recommendations may be warranted in event the site soils exposed during construction appear different from those as described earlier in this report.

Recommendations provided are based on assumptions that structural footings will be established exclusively into compacted engineered fills. No footings and/or slabs are allowed straddling over cut/fill transition interface.

FOOTING TRENCH EXCAVATIONS AND SLAB SUBGRADES SHALL BE VERIFIED IMMEDIATELY PRIOR TO VAPORT BARRIER COVERING AND CONCRETE POUR. SOILS SOUTHWEST WILL ASSUME NO RESPONSIBILITY OF ANY FUTURE STRUCTURAL DISTRESS IN EVENT THE ABOVE CONDITIONS ARE NOT MET.

This office should review final grading and foundation plans when they become available. Footing excavations should be inspected prior to steel and concrete placement to ensure that foundations are founded into satisfactory soils and excavations are free of loose and disturbed materials. Similar subgrade verifications are recommended prior to concrete slab-on-grade placement.

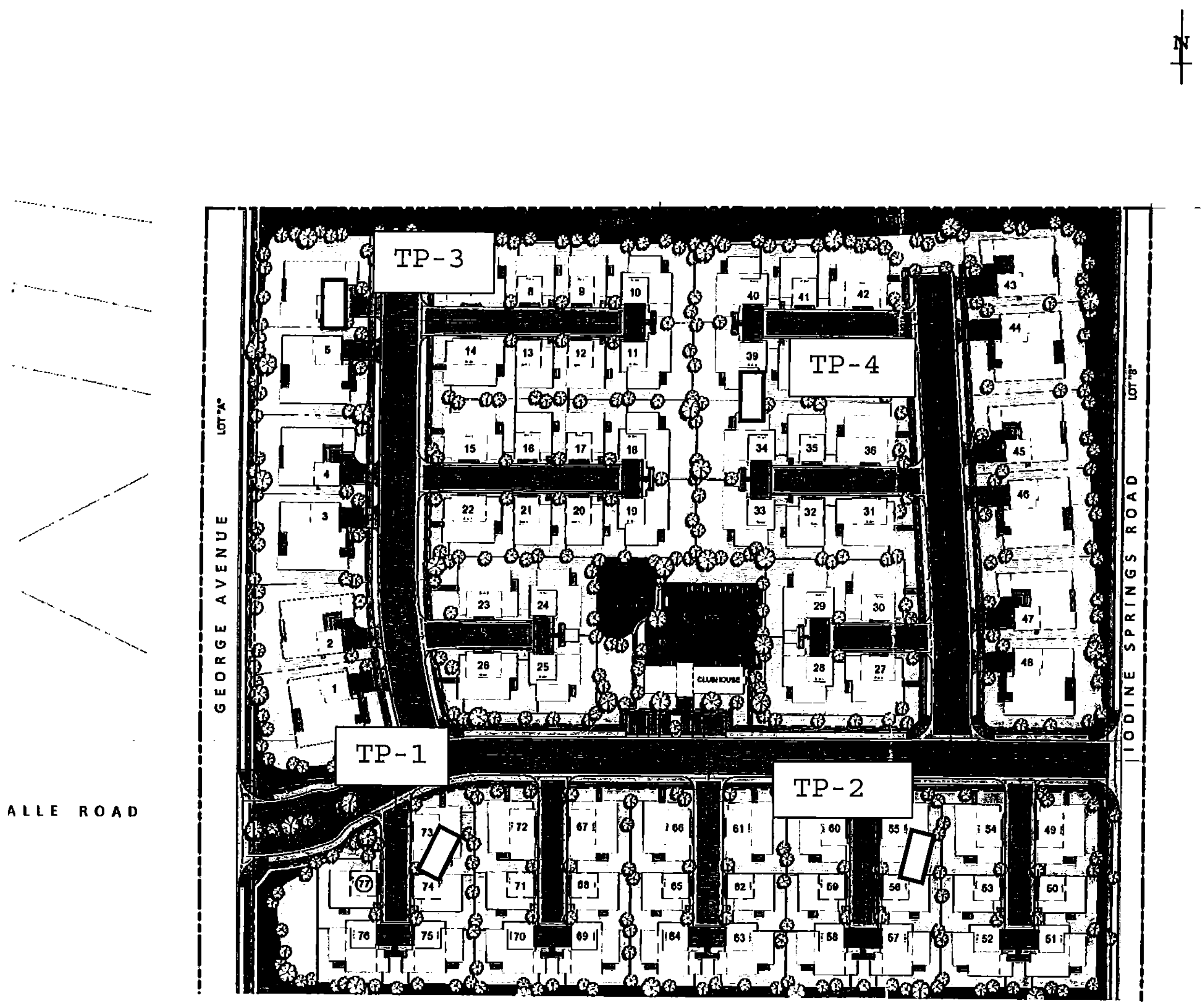
A pregrading meeting between grading contractor and soils engineer is recommended prior to construction preferably at the site, to discuss the grading procedures to be implemented and other requirements described in this report to be fulfilled.

This report has been prepared exclusively for the use of the addressee for the project referenced in the context. It shall not be transferred or be used by other parties without a written consent by Soils Southwest, Inc. We cannot be responsible for use of this report by others without the necessary inspection and testing by our personnel.

Should the project be delayed beyond one year after the date of this report; the recommendations presented shall be reviewed to consider any possible change in site conditions.

The recommendations presented are based on the assumption that a representative of this office will perform the necessary geotechnical observations and testing during construction. The field observations are considered a continuation of the geotechnical investigation performed. If another firm is retained for geotechnical observations and testing, our professional liability and responsibility shall be limited to the extent that Soils Southwest, Inc. would not be the geotechnical engineer of record and a letter of Transfer of Responsibility should be provided accordingly.

PLOT PLAN AND TEST LOCATIONS
(Schematic, Not To Scale)




Legend ▭ TP-1 Approximate Test Trench Explorations Plate 1

# 7.0 APPENDIX A

## Field Explorations

Field evaluations included site reconnaissance and four (4) test trenches using a backhoe. During site reconnaissance, the surface conditions were noted and test exploration locations were determined.

Soils encountered during explorations were logged and such were classified by visual observations in accordance with the generally accepted classification system. The field descriptions were modified, where appropriate, to reflect laboratory test results. Approximate test locations are shown on Plate 1.

Relatively undisturbed soils were sampled using a drive sampler lined with soil sampling rings. The split barrel steel sampler was driven into the bottom of test excavations at various depths. Soil samples were retained in brass rings of 2.5 inches in diameter and 1.00 inch in height. The central portion of each sample was enclosed in a close-fitting waterproof container for shipment to our laboratory. In addition to undisturbed sample, bulk soil samples were procured as described in the logs.

Logs of test explorations are presented in the following summary sheets that include the description of the soils and/or fill materials encountered.

# LOG OF TEST EXPLORATIONS

Soils Southwest, Inc.
897 Via Lata, Suite N
Colton, CA 92324
(909) 370-0474 Fax (909) 370-3156

# LOG OF TEST PIT TP-1

| | | | |
|---|---|---|---|
| **Project:** Marimina, LLC / Nova Homes, Inc. | | **Job No.:** 15021-F/BMP | |
| **Logged By:** John F. | **Boring Diam.:** 18" Bucket | **Date:** June 30, 2015 | |

| Sample Type | Water Content in % | Dry Density in PCF | Percent Compaction | Unified Classification System | Graphic | Depth in Feet | Description and Remarks |
|---|---|---|---|---|---|---|---|
| | | | | SP-SM | | | SOUTHWEST<br>weeds |
| | | | | | | | SAND - dense gray brown to dark gray, semi-cemented fine to medium with silts, pebble,loose, damp |
| California sampler | 6.0 | 113 | 86 | SP | | | |
| Bulk/Grab sample | | | | | | 5 | - Well cemented gravely gray-brown fine to coarse with small piholes, pebbles scattered rock fragments, damp |
| | | | | | | | - No sample recovery |
| | | | | | | | - color change to pale yellow gray brown, very dense well cemented slightly silty, traces of caliche fine, damp, small pinholes |
| | | | | | | 10 | - very dense, dg, fine to coarse with silts, rippible soft granite, damp |
| | | | | SM | | | SILTY SAND - fine, moist |
| | | | | | | | - End of test trench @ 12.0 ft.<br>- no bedrock<br>- no groundwater |
| | | | | | | 15 | |
| | | | | | | 20 | |
| | | | | | | 25 | |
| | | | | | | 30 | |

**Groundwater:** n/a
**Approx. Depth of Bedrock:** n/a
**Datum:** n/a
**Elevation:** n/a

**Site Location**

35775 Iodine Springs Road n/o
Clinton Keith Rd.
Wildomar, California

**Plate #**

■ California sampler ◥ Bulk/Grab sample

Soils Southwest, Inc.
897 Via Lata, Suite N
Colton, CA 92324
(909) 370-0474 Fax (909) 370-3156

# LOG OF TEST PIT TP-2

**Project:** Marimina, LLC / Nova Homes, Inc.
**Job No.:** 15021-F/BMP
**Logged By:** John F.
**Boring Diam.:** 18" Bucket
**Date:** June 30, 2015

| Sample Type | Water Content in % | Dry Density in PCF | Percent Compaction | Unified Classification System | Graphic | Depth in Feet | Description and Remarks |
|---|---|---|---|---|---|---|---|
| | | | | SP-SM | | | SOUTHEAST<br>weeds |
| | | | | | | | SAND - gray brown to dark gray, slightly silty, semi-cemented, fine to medium, pebbles, loose, damp |
| Bulk/Grab; California sampler | 7.5 | 114 | 87 | SP | | 5 | - color change to pale yellow brown, cemented, dense, fine to medium coarse, pebbles |
| | | | | | | | - color change to pale yellow gray-brown to white, well-cemented, traces of caliche, silts, fine to medium coarse, very dense, dry. slighty porous (small pinholes). |
| California sampler | | | | | | 10 | - fine to coarse gravely with rock fragments, very dense, damp |
| | | | | | | | - color change to light brown, traces of silt, gravely, coarse to very coarse, moist |
| | | | | | | 15 | - * NR<br>- End of test boring @ 8.0 ft.<br>- no bedrock<br>- no groundwater<br><br>* NR = No Recovery |
| | | | | | | 20 | |
| | | | | | | 25 | |
| | | | | | | 30 | |

**Groundwater:** n/a
**Approx. Depth of Bedrock:** n/a
**Datum:** n/a
**Elevation:** n/a

**Site Location**

35775 Iodine Springs Road n/o
Clinton Keith Rd.
Wildomar, California

**Plate #**

■ California sampler ◣ Bulk/Grab sample

**Soils Southwest, Inc.**
897 Via Lata, Suite N
Colton, CA 92324
(909) 370-0474 Fax (909) 370-3156

# LOG OF TEST PIT TP-3

**Project:** Marimina, LLC / Nova Homes, Inc. **Job No.:** 15021-F/BMP
**Logged By:** John F. **Boring Diam.:** 18" Bucket **Date:** June 30, 2015

| Sample Type | Water Content in % | Dry Density in PCF | Percent Compaction | Unified Classification System | Graphic | Depth in Feet | Description and Remarks |
|---|---|---|---|---|---|---|---|
| Bulk/Grab | | | | SP-SM | | | NORTHWEST<br>weeds, rock, cobbles, scattered boulders |
| | | | | | | | SAND - brown, slightly silty, fine to medium coarse, pebble, rock fragments |
| California | 10 | 119 | 91 | GP-SP | | | - color change to light gray, gravely, medium to coarse, dg, rippible granite, dry |
| | | | | | | 5 | |
| | | | | | | | - color change to yellowish light brown very dense, decomposed granite, rippible granite, dry to damp |
| | | | | | | 10 | - Abandoned test trench @ 3.5 ft. due to resistance (very dense granitic material)<br>- bedrock @ +/- 3.0 ft.<br>- no groundwater |
| | | | | | | 15 | |
| | | | | | | 20 | |
| | | | | | | 25 | |
| | | | | | | 30 | |

**Groundwater:** n/a
**Approx. Depth of Bedrock:** n/a
**Datum:** n/a
**Elevation:** n/a

**Site Location**
35775 Iodine Springs Road n/o
Clinton Keith Rd.
Wildomar, California

**Plate #**

■ California sampler ◣ Bulk/Grab sample

Soils Southwest, Inc.
897 Via Lata, Suite N
Colton, CA 92324
(909) 370-0474 Fax (909) 370-3156

# LOG OF TEST PIT TP-4

| | | | |
|---|---|---|---|
| **Project:** Marimina, LLC / Nova Homes, Inc. | | **Job No.:** | 15021-F/BMP |
| **Logged By:** John F. | **Boring Diam.:** 18" Bucket | **Date:** | June 30, 2015 |

| Sample Type | Water Content in % | Dry Density in PCF | Percent Compaction | Unified Classification System | Graphic | Depth in Feet | Description and Remarks |
|---|---|---|---|---|---|---|---|
| | | | | SP-SM | | | NORTH CENTER<br>weeds |
| Bulk/Grab sample | | | | | | | SAND - light gray brown, gravely, slightly silty, fine to medium coarse, rock fragments, dry |
| | | | | | | 5 | - color change to dark gray to black, slightly silty, traces of clay, fine moist |
| California sampler | | | | SP | | | - color change to light brown, slightly silty, fine to medium coarse pebbles<br>- (Max Density =131 pcf @ 8.5%) |
| | | | | GP-SP | | 10 | - color change to gray, traces of silt, fine to medium coarse, decomposed granite, rippible granite, dry<br>- * NR<br>- color change to gray-brown with traces of clay, fine to medium, pebbles |
| | | | | | | 15 | - gravely medium coarse, decomposed granite, rippible granite<br>- End of test trench @ 8.5 ft.<br>- bedrock @ +/- 8.0 ft.<br>- no groundwater<br>* NR = No recovery |
| | | | | | | 20 | |
| | | | | | | 25 | |
| | | | | | | 30 | |

**Groundwater:** n/a
**Approx. Depth of Bedrock:** n/a
**Datum:** n/a
**Elevation:** n/a

**Site Location**

35775 Iodine Springs Road n/o
Clinton Keith Rd.
Wildomar, California

**Plate #**

■ California sampler ◣ Bulk/Grab sample

# 8.0 APPENDIX B

## Laboratory Test Programs

Laboratory tests were conducted on representative soils for the purpose of classification and for the determination of the physical properties and engineering characteristics. The number and selection of the types of testing for a given study are based on the geotechnical conditions of the site. A summary of the various laboratory tests performed for the project is presented below.

Moisture Content and Dry Density (D2937):

Data obtained from these test, performed on undisturbed samples are used to aid in the classification and correlation of the soils and to provide qualitative information regarding soil strength and compressibility.

Direct Shear (D3080):

Data obtained from this test performed at increased and field moisture conditions on relatively remolded soil sample is used to evaluate soil shear strengths. Samples contained in brass sampler rings, placed directly on test apparatus are sheared at a constant strain rate of 0.002 inch per minute under saturated conditions and under varying loads appropriate to represent anticipated structural loadings. Shearing deformations are recorded to failure. Peak and/or residual shear strengths are obtained from the measured shearing load versus deflection curve. Test results, plotted on graphical form, are presented on Plate B-1 of this section.

Consolidation (D2835):

Drive-tube samples are tested at their field moisture contents and at increased moisture conditions since the soils may become saturated during life-time use of the planned structure.

Data obtained from this test performed on relatively undisturbed and/or remolded samples, were used to evaluate the consolidation characteristics of foundation soils under anticipated foundation loadings. Preparation for this test involved trimming the sample, placing it in one inch high brass ring, and loading it into the test apparatus which contained porous stones to accommodate drainage during testing. Normal axial loads are applied at a load increment ratio, successive loads being generally twice the preceding.

Soil samples are usually under light normal load conditions to accommodate seating of the apparatus. Samples were tested at the field moisture conditions at a predetermined normal load. Potentially moisture sensitive soil typically demonstrated significant volume change with the introduction of free water. The results of the consolidation tests are presented in graphical forms on Plate B-2.

## Laboratory Test Results

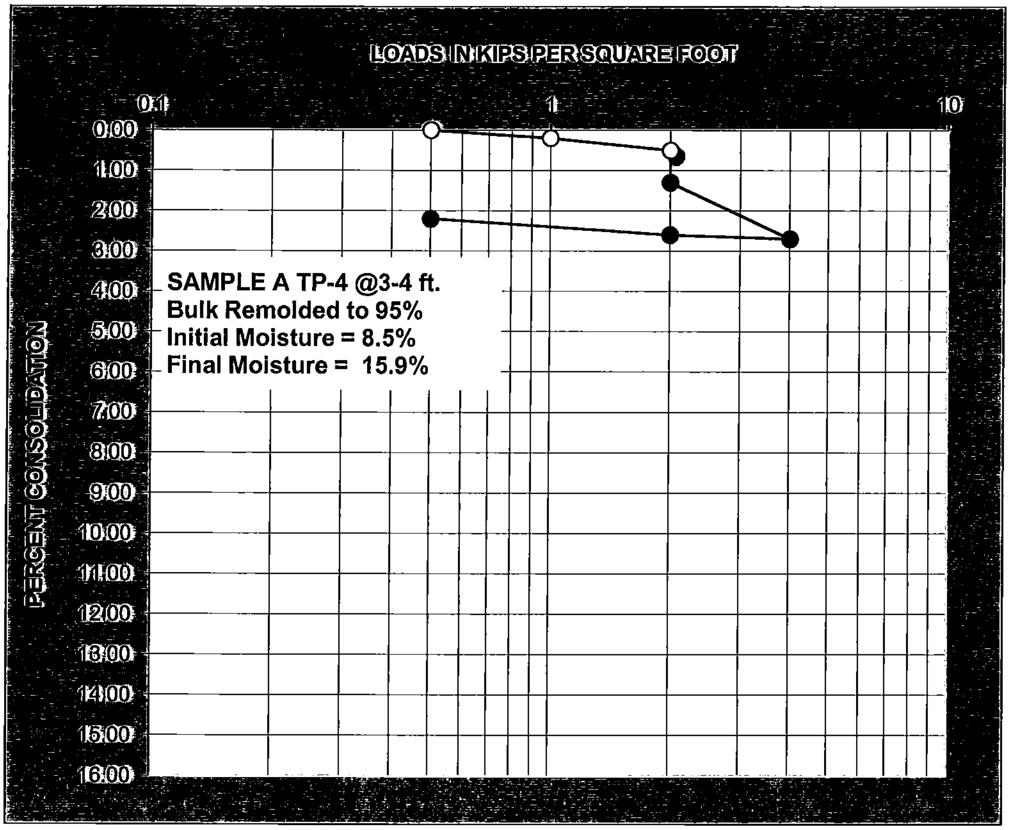
A. Table I: In-Situ Moisture-Density (ASTM D2937)

| Test Boring No. | Sample Depth, ft. | Dry Density, pcf. | Moisture Content, % |
|---|---|---|---|
| 1 | 3.0 | 113.3 | 7.69 |

B. Table II: Max. Density/Optimum Moisture Content (ASTM D1557)

| Sample Location | Max. Dry Density, pcf | Opt. Moisture (%) |
|---|---|---|
| TP-1 @ 3-4 ft. | 131 | 8.5 |

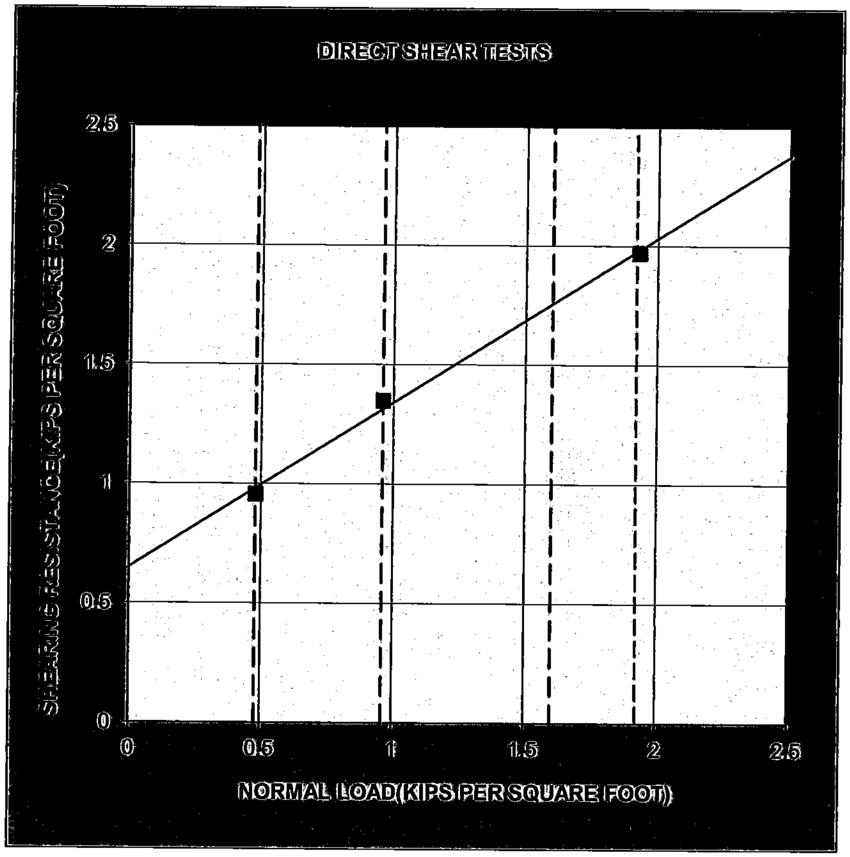
# CONSOLIDATION TESTS




- **WATER PERMITTED TO CONTACT SAMPLE**

| | | | | |
|---|---|---|---|---|
| PROJECT | Marimina LLC / Nova Homes, Inc.<br>35775 Iodine Springs Road, Wildomar | | | |
| PROJECT NO. | 15021-F | PLATE | B-2b | |

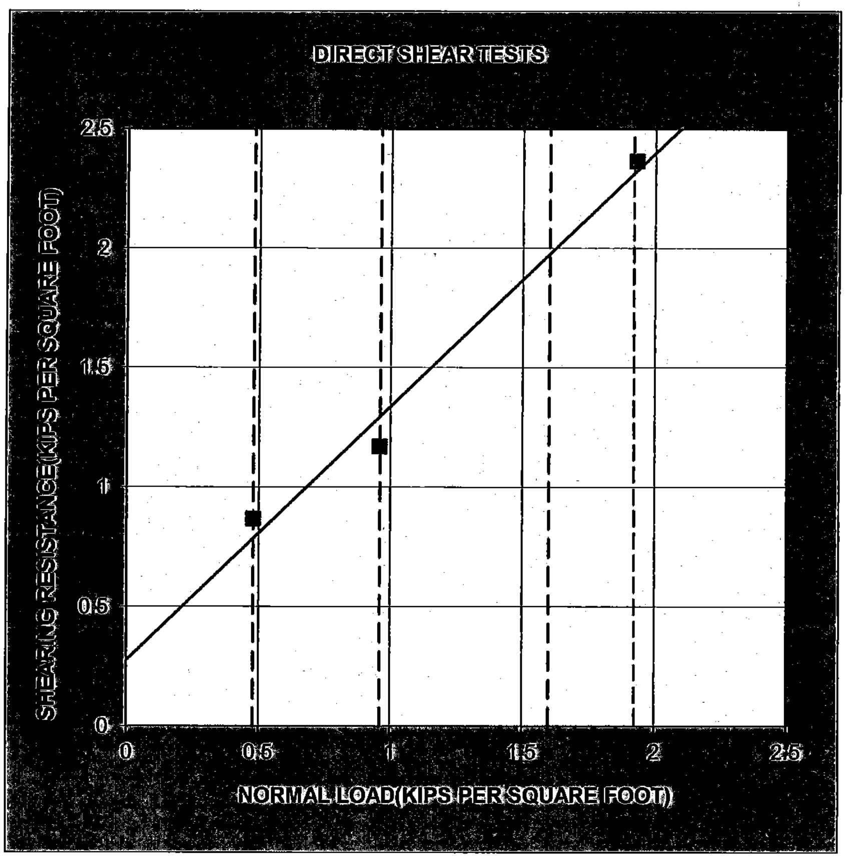
**SOILS SOUTHWEST INC.**
**Consulting Foundation Engineers**




| SYMBOL | LOCATION | DEPTH (FT) | TEST CONDITION | COHESION (psf) | FRICTION (degree) |
|---|---|---|---|---|---|
| ■ | TP-4 | 3 to 4 | Remolded to 90% Residual | 650.08 | 34.63 |

| | | |
|---|---|---|
| Marimina, LLC / Nova Homes, Inc.<br>35775 Iodine Springs Road<br>Wildomar, California | PROJECT NO. | 15021-F |
| | PLATE | B-1a |

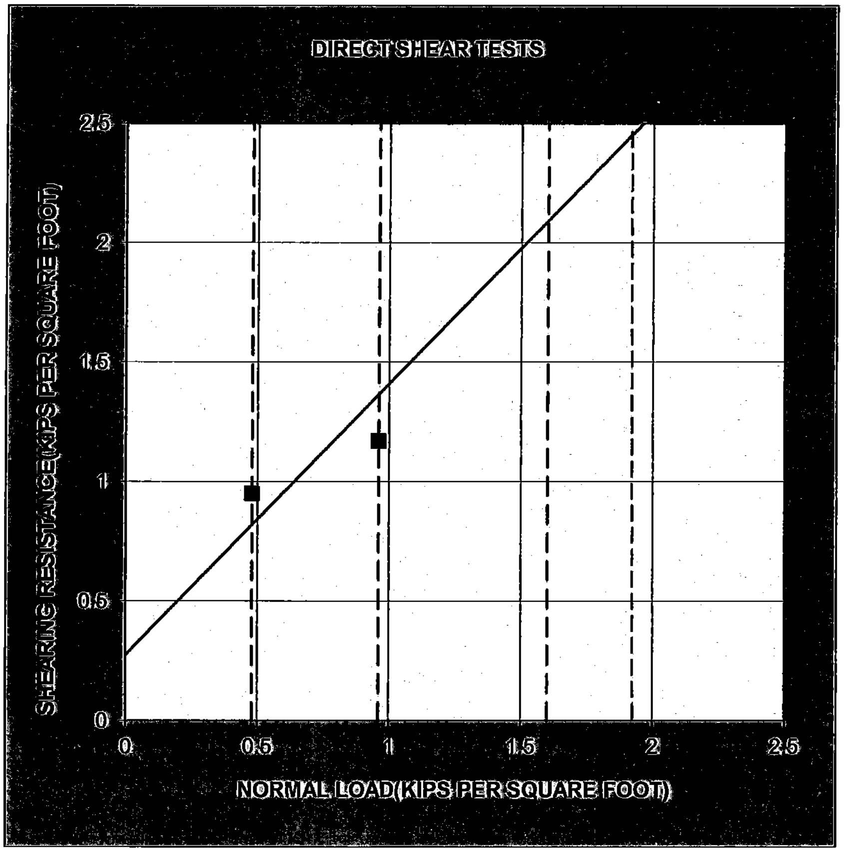
**SOILS SOUTHWEST, INC.**
**Consulting Foundation Engineers**




| SYMBOL | LOCATION | DEPTH (FT) | TEST CONDITION | COHESION (psf) | FRICTION (degree) |
|---|---|---|---|---|---|
| ■ | TP-4 | 3 to 4 | Remolded to 95% Residual | 275.38 | 46.71 |

| | | |
|---|---|---|
| **Marimina, LLC / Nova Homes, Inc.**<br>**35775 Iodine Springs Road**<br>Wildomar, California | PROJECT NO. | 15021-F |
| | PLATE | B-1-1a |

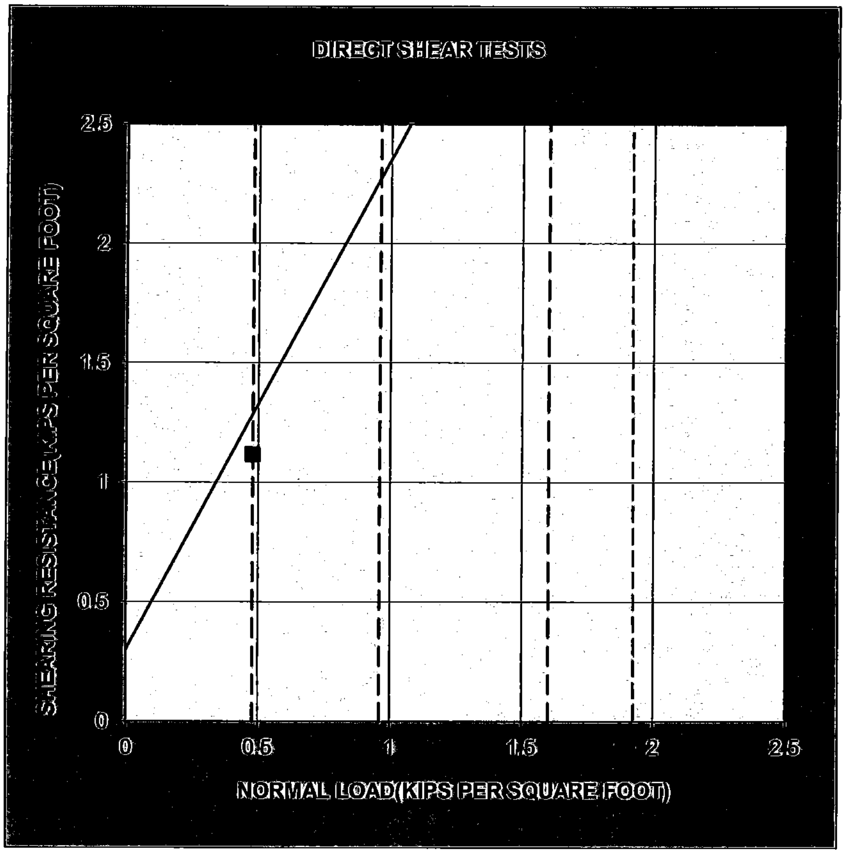
**SOILS SOUTHWEST, INC.**
**Consulting Foundation Engineers**




| SYMBOL | LOCATION | DEPTH (FT) | TEST CONDITION | COHESION (psf) | FRICTION (degree) |
|---|---|---|---|---|---|
| ■ | TP-4 | 3 to 4 | Remolded to 95% Peak | 275.00 | 48.62 |

| | | |
|---|---|---|
| Marimina, LLC / Nova Homes, Inc.<br>35775 Iodine Springs Road<br>Wildomar, California | PROJECT NO. | 15021-F |
| | PLATE | B-1-1b |

**SOILS SOUTHWEST, INC.**
**Consulting Foundation Engineers**

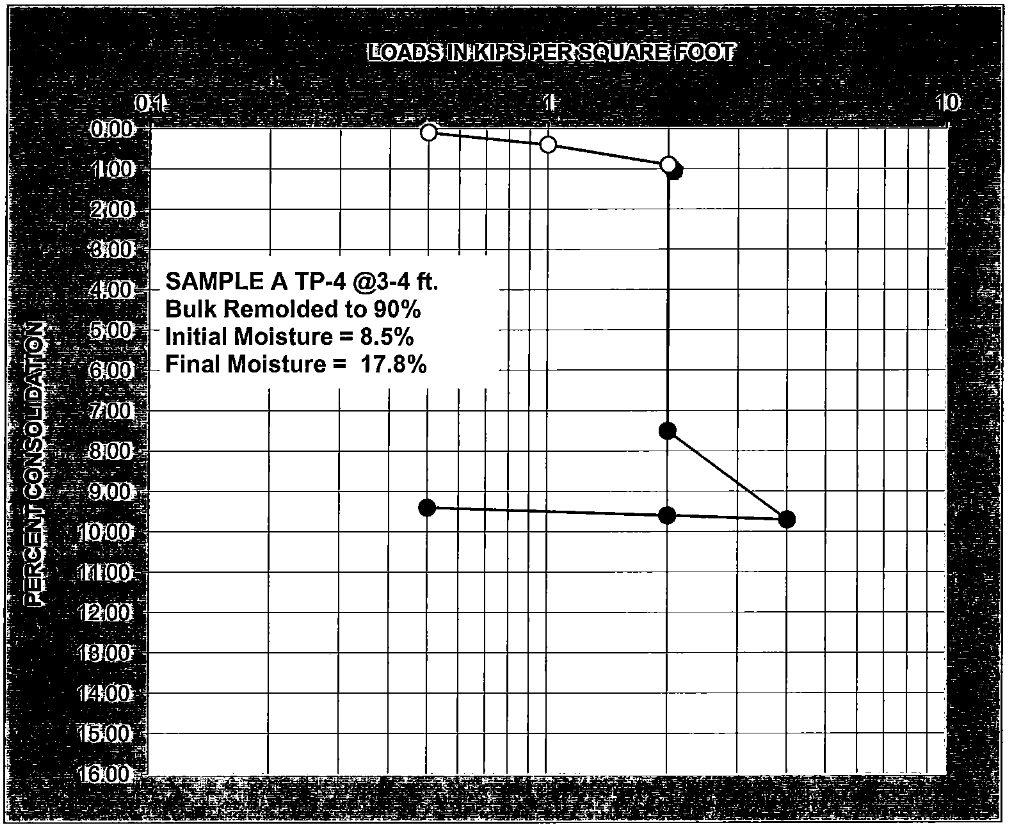



| SYMBOL | LOCATION | DEPTH (FT) | TEST CONDITION | COHESION (psf) | FRICTION (degree) |
|---|---|---|---|---|---|
| ■ | TP-1 | 3 to 5 | Remolded to 90% Residual | 300.37 | 64.02 |

| | | |
|---|---|---|
| **Marimina, LLC / Nova Homes, Inc.**<br>**35775 Iodine Springs Road**<br>Wildomar, California | PROJECT NO. | 15021-F |
| | PLATE | B-1-1 |

**SOILS SOUTHWEST, INC.**
**Consulting Foundation Engineers**

# CONSOLIDATION TESTS

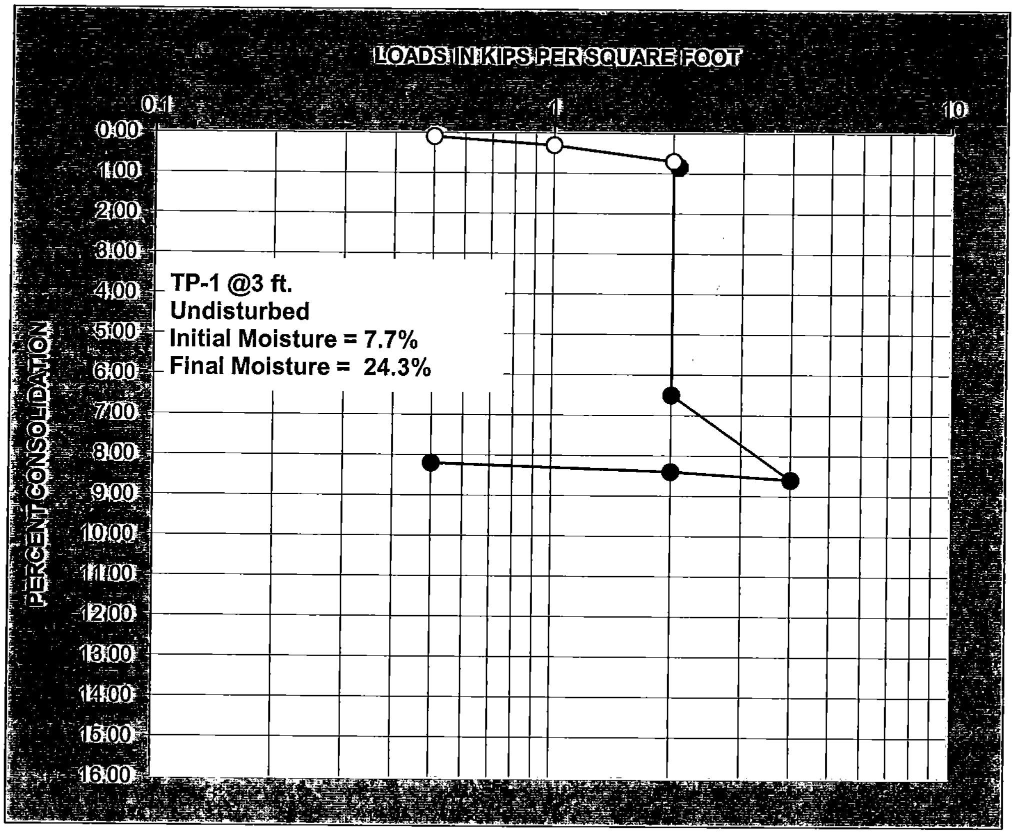



- **WATER PERMITTED TO CONTACT SAMPLE**

| | | | | |
|---|---|---|---|---|
| PROJECT | Marimina LLC / Nova Homes, Inc.<br>35775 Iodine Springs Road, Wildomar | | | |
| PROJECT NO. | 15021-F | PLATE | B-2 | |

## SOILS SOUTHWEST INC.
**Consulting Foundation Engineers**

## CONSOLIDATION TESTS




● **WATER PERMITTED TO CONTACT SAMPLE**

| | | | | |
|---|---|---|---|---|
| PROJECT | Marimina LLC / Nova Homes, Inc.<br>35775 Iodine Springs Road, Wildomar | | | |
| PROJECT NO. | 15021-F | PLATE | B-2-1 | |

**SOILS SOUTHWEST INC.**
**Consulting Foundation Engineers**

# APPENDIX C

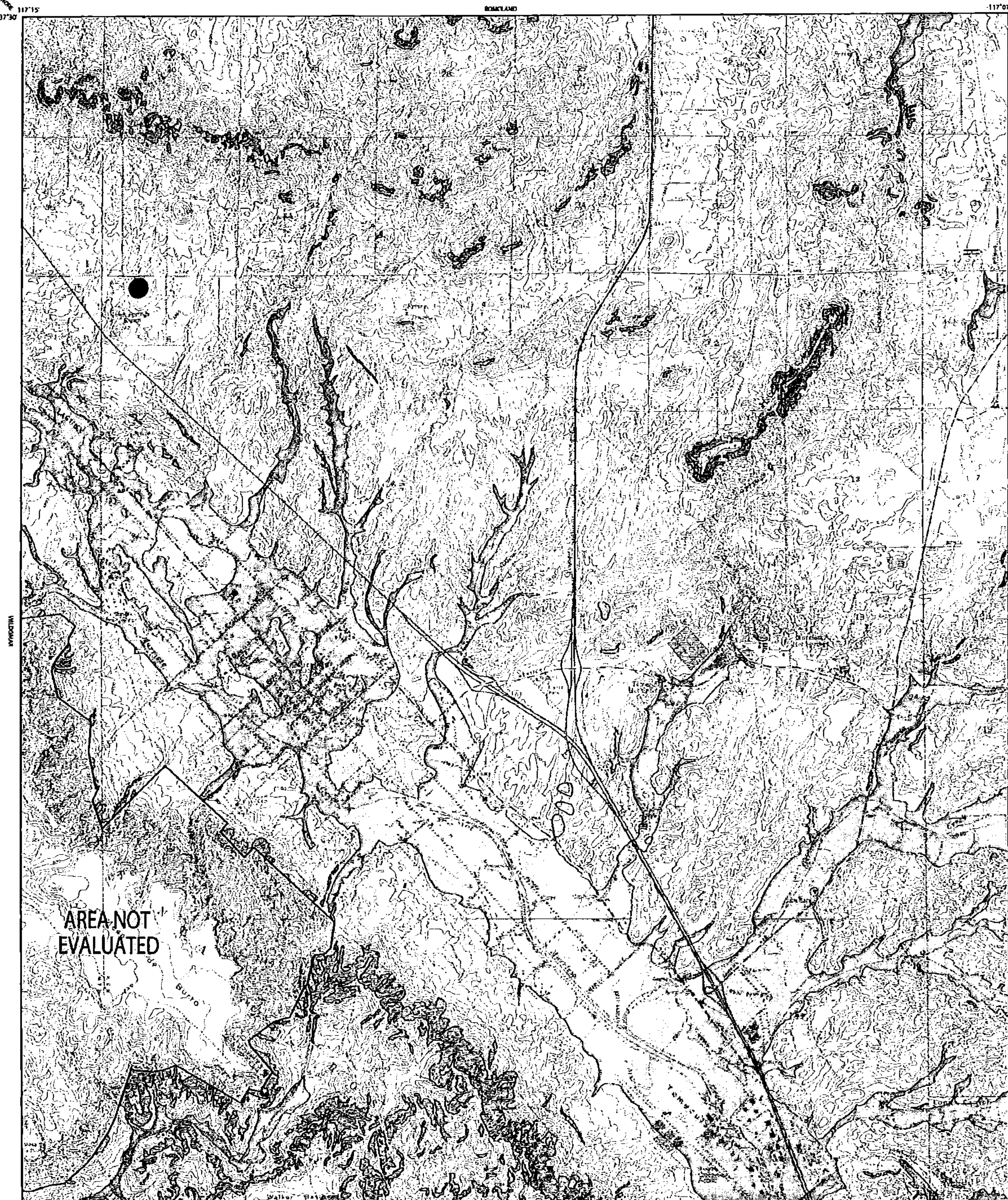
Supplemental Seismic Design Parameters

As per 2013 CBC

CALIFORNIA GEOLOGICAL SURVEY
JOHN G. PARRISH, PH.D., STATE GEOLOGIST

STATE OF CALIFORNIA-ARNOLD SCHWARZENEGGER, GOVERNOR
THE RESOURCES AGENCY-MIKE CHRISMAN, SECRETARY
DEPARTMENT OF CONSERVATION-BRIDGETT LUTHER, DIRECTOR

SEISMIC HAZARD ZONES
Murrieta Quadrangle




Base Map prepared by U.S. Geological Survey, 1953, photorevised 1979.
Zones of required investigation boundaries may reflect updated digital topographic data that can differ significantly from contours shown on the base map.

## PURPOSE OF MAP

This map will assist cities and counties in fulfilling their responsibilities for protecting the public from the effects of earthquake-triggered ground failure as required by the Seismic Hazards Mapping Act (Public Resources Code Sections 2690-2699.6).

For information regarding the general approach and recommended methods for preparing this map, see California Geological Survey Special Publication 118, *Recommended Criteria for Delineating Seismic Hazard Zones in California*.

For information regarding the scope and recommended methods to be used in conducting the required site investigations, see California Geological Survey Special Publication 117, *Guidelines for Evaluating and Mitigating Seismic Hazards in California*.

For a general description of the Seismic Hazard Mapping Program, the Seismic Hazards Mapping Act and regulations, and related information, please refer to the website at www.conservation.ca.gov/cgs/.

1) This map may not show all areas that have the potential for liquefaction, landsliding, strong earthquake ground shaking or other earthquake and geologic hazards. Also, a single earthquake capable of causing liquefaction or triggering landslide failure will not uniformly affect the entire area zoned.

2) Liquefaction zones may also contain areas susceptible to the effects of earthquake-induced landslides. This situation typically exists at or near the toe of existing landslides, downslope from rockfall or debris flow source areas, or adjacent to steep stream banks.

3) This map does not show Alquist-Priolo earthquake fault zones, if any, that may exist in this area. Please refer to the latest official map of earthquake fault zones for disclosures and other actions that are required by the Alquist-Priolo Earthquake Fault Zoning Act. For more information on this subject and an index to available maps, see DMG Special Publication 42.

4) Landslide zones on this map were determined, in part, by adapting methods originally developed by the U.S. Geological Survey (USGS). Landslide hazard maps prepared by the USGS typically use experimental approaches to assess earthquake-induced and other types of landslide hazards. Although aspects of these new methodologies may be incorporated in future California Geological Survey (CGS) seismic hazard zone maps, USGS maps should not be used as substitutes for these Official SEISMIC HAZARD ZONES maps.

5) U.S. Geological Survey base map standards provide that 90 percent of cultural features be located within 40 feet (horizontal accuracy) at the scale of this map. The identification and location of liquefaction and earthquake-induced landslide zones are based on available data. However, the quality of data used is varied. The zone boundaries depicted have been drawn as accurately as possible at this scale.

6) Information on this map is not sufficient to serve as a substitute for the geologic and geotechnical site investigations required under Chapters 7.5 and 7.8 of Division 2 of the Public Resources Code.

7) DISCLAIMER: The State of California and the Department of Conservation make no representations or warranties regarding the accuracy of the data from which these maps were derived. Neither the State nor the Department shall be liable under any circumstances for any direct, indirect, special, incidental or consequential damages with respect to any claim by any user or any third party on account of or arising from the use of this map.

SCALE 1:24,000

STATE OF CALIFORNIA

# California Geological Survey

## SEISMIC HAZARD ZONES

Delineated in compliance with
Chapter 7.8, Division 2 of the California Public Resources Code
(Seismic Hazards Mapping Act)

## MURRIETA QUADRANGLE

OFFICIAL MAP

Released: December 5, 2007

STATE GEOLOGIST

## MAP EXPLANATION

Zones of Required Investigation:

**Liquefaction**

Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

**Earthquake-Induced Landslides**

Areas where previous occurrence of landslide movement, or local topographic, geologic, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

NOTE: Seismic Hazard Zones identified on this map may include developed land where delineated hazards have already been mitigated to city or county standards. Check with your local building/planning department for information regarding the location of such mitigated areas.

DATA AND METHODOLOGY USED TO DEVELOP
THIS MAP ARE PRESENTED IN THE FOLLOWING:

Seismic Hazard Zone Report of the Murrieta 7.5-minute Quadrangle, Riverside County, California: California Geological Survey, Seismic Hazard Zone Report 115.

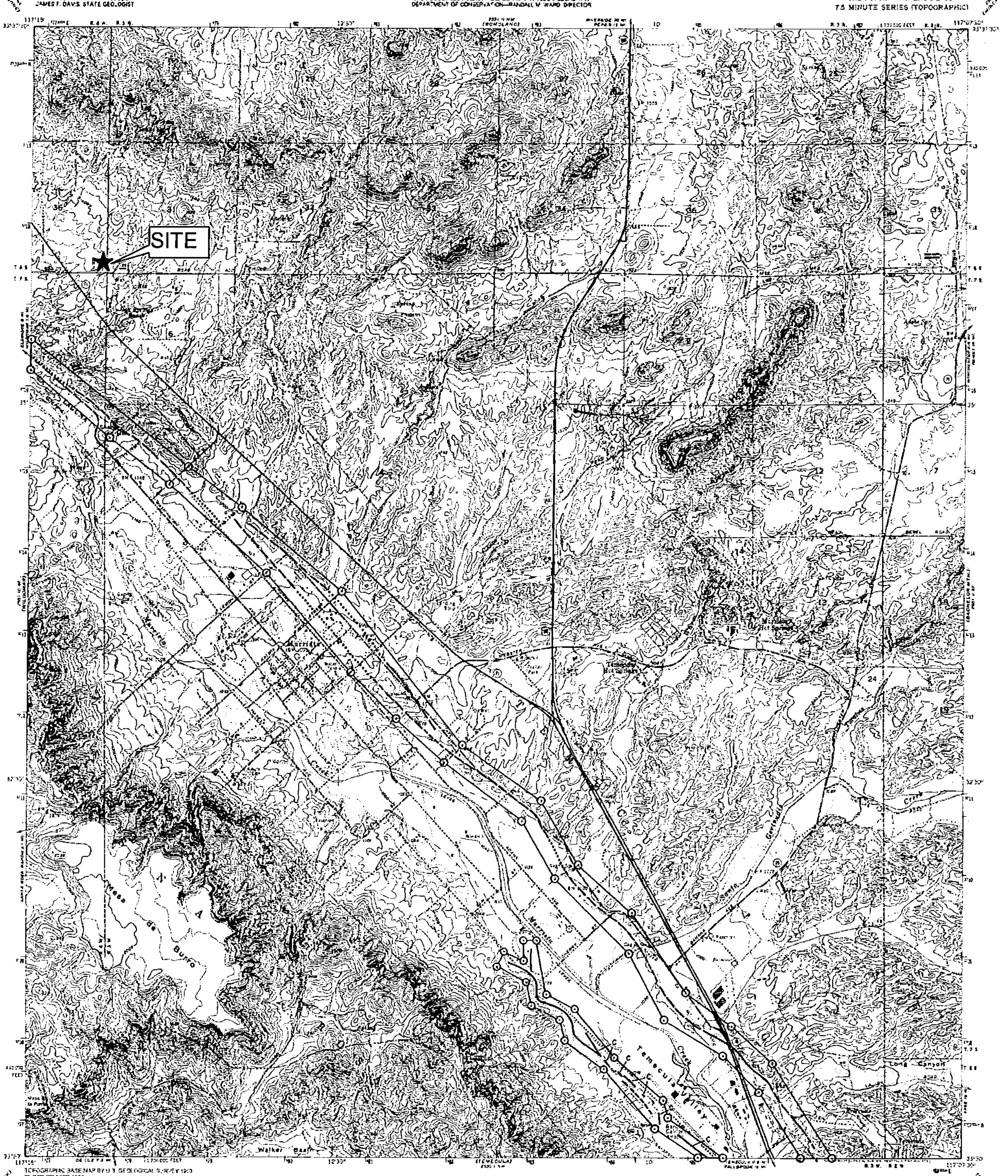
For additional information on seismic hazards in this map area, the rationale used for zoning, and additional references consulted, refer to:
www.conservation.ca.gov/cgs/shzp/

California Geological Survey
Geologic Information and Publications
801 K Street, MS 14-34
Sacramento, CA 95814-3532
(916) 445-5716
www.conservation.ca.gov/cgs

DIVISION OF MINES AND GEOLOGY
JAMES F. DAVIS, STATE GEOLOGIST

STATE OF CALIFORNIA—GEORGE DEUKMEJIAN, GOVERNOR
THE RESOURCES AGENCY—GORDON K. VAN VLECK, SECRETARY
DEPARTMENT OF CONSERVATION—RANDALL M. WARD, DIRECTOR

MURRIETA QUADRANGLE
CALIFORNIA—RIVERSIDE CO.
7.5 MINUTE SERIES (TOPOGRAPHIC)




TOPOGRAPHIC BASE MAP BY U.S. GEOLOGICAL SURVEY 1953
PHOTO REVISED 1973

Mapped, edited, and published by the Geological Survey

Control by USGS, USC&GS, and USCE

Topography from aerial photographs by multiplex methods and by plane-table surveys 1953. Aerial photographs taken 1951

Polyconic projection. 1927 North American datum
10,000-foot grid based on California coordinate system, zone 6

Dashed land lines indicate approximate locations

Unchecked elevations are shown in brown

1000-meter Universal Transverse Mercator grid ticks, zone 11, shown in blue

SCALE 1:24,000

CONTOUR INTERVAL 20 FEET
DASHED LINES REPRESENT 10-FOOT CONTOURS
DATUM IS MEAN SEA LEVEL

THIS MAP COMPLIES WITH NATIONAL MAP ACCURACY STANDARDS

ROAD CLASSIFICATION

Heavy-duty — Light-duty — Medium-duty — Unimproved dirt

U.S. Route — State Route

QUADRANGLE LOCATION

# STATE OF CALIFORNIA
# SPECIAL STUDIES ZONES

**Delineated in compliance with**
**Chapter 7.5, Division 2 of the California Public Resources Code**
*(Alquist-Priolo Special Studies Zones Act)*

*Murrieta Quadrangle*

## REVISED OFFICIAL MAP

Effective: January 1, 1990

[signature] State Geologist

## MAP EXPLANATION

**Active Faults**

Faults considered to have been active during Holocene time and to have a relatively high potential for surface rupture; solid line where accurately located, long dash where approximately located, short dash where inferred, dotted where concealed; query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by creep or possible creep.

**Special Studies Zone Boundaries**

These are delineated as straight-line segments that connect encircled turning points so as to define special studies zone segments.

Seaward projection of zone boundary.

**REFERENCES USED TO COMPILE FAULT DATA**

Murrieta Quadrangle


Kennedy, M.P., 1977, Recency and character of faulting along the Elsinore fault zone in southern Riverside County, California: California Division of Mines and Geology Special Report 131, 12 p.

[illegible]

[illegible]


[illegible]

**IMPORTANT – PLEASE NOTE**

1) *This map may not show all faults that have the potential for surface fault rupture, either within the special studies zones or outside their boundaries.*
2) Faults shown are the basis for establishing the boundaries of the special studies zones.
3) The identification and location of these faults are based on the best available data. However, the quality of data used is varied. Traces have been drawn as accurately as possible at this map scale.
4) Fault information on this map is not sufficent to serve as a substitute for the geologic site investigations (special studies) required under Chapter 7.5 of Division 2 of the California Public Resources Code.

USGS
science for a changing world

**Earthquake Hazards Program**

# 2008 National Seismic Hazard Maps - Source Parameters

Output Selected Faults (Excel)

| Output | Distance in Kilometers | Name | St | Fault parallel slip rate | Preferred Dip (degrees) | Dip Dir | Slip Sense | Rupture Top (km) | Rupture Bottom (km) | Length (km) |
|---|---|---|---|---|---|---|---|---|---|---|
| ☑ | 1.56 | Elsinore;GI+T | CA5 | | 90 | V | strike slip | 0 | | 78 |
| ☑ | 1.56 | Elsinore;GI+T+J | CA | | 86 | NE | strike slip | 0 | | 153 |
| ☑ | 1.56 | Elsinore;GI+T+J+CM | CA | | 86 | NE | strike slip | 0 | | 195 |
| ☑ | 1.56 | Elsinore;T | CA5 | | 90 | V | strike slip | 0 | | 52 |
| ☑ | 1.56 | Elsinore;T+J | CA | | 86 | NE | strike slip | 0 | | 127 |
| ☑ | 1.56 | Elsinore;T+J+CM | CA | | 85 | NE | strike slip | 0 | | 169 |
| ☑ | 1.56 | Elsinore;W+GI+T | CA | | 84 | NE | strike slip | 0 | | 124 |
| ☑ | 1.56 | Elsinore;W+GI+T+J | CA | | 84 | NE | strike slip | 0 | | 200 |
| ☑ | 1.56 | Elsinore;W+GI+T+J+CM | CA | | 84 | NE | strike slip | 0 | | 242 |
| ☑ | 4.44 | Elsinore;GI | CA5 | | 90 | V | strike slip | 0 | | 37 |
| ☑ | 4.44 | Elsinore;W+GI | CA | | 81 | NE | strike slip | 0 | | 83 |
| ☑ | 30.96 | San Jacinto;A | CA9 | | 90 | V | strike slip | 0 | | 71 |
| ☑ | 30.96 | San Jacinto;A+C | CA | | 90 | V | strike slip | 0 | | 118 |
| ☑ | 30.96 | San Jacinto;A+CC | CA | | 90 | V | strike slip | 0 | | 118 |
| ☑ | 30.96 | San Jacinto;A+CC+B | CA | | 90 | V | strike slip | 0.1 | | 152 |
| ☑ | 30.96 | San Jacinto;A+CC+B+SM | CA | | 90 | V | strike slip | 0.1 | | 178 |
| ☑ | 32.55 | San Jacinto;SBV+SJV+A | CA | | 90 | V | strike slip | 0 | | 134 |
| ☑ | 32.55 | San Jacinto;SBV+SJV+A+C | CA | | 90 | V | strike slip | 0 | | 181 |
| ☑ | 32.55 | San Jacinto;SBV+SJV+A+CC | CA | | 90 | V | strike slip | 0 | | 181 |
| ☑ | 32.55 | San Jacinto;SBV+SJV+A+CC+B | CA | | 90 | V | strike slip | 0.1 | | 215 |
| ☑ | 32.55 | San Jacinto;SBV+SJV+A+CC+B+SM | CA | | 90 | V | strike slip | 0.1 | | 242 |
| ☑ | 32.55 | San Jacinto;SJV+A | CA | | 90 | V | strike slip | 0 | | 89 |
| ☑ | 32.55 | San Jacinto;SJV+A+C | CA | | 90 | V | strike slip | 0 | | 136 |
| ☑ | 32.55 | San Jacinto;SJV+A+CC | CA | | 90 | V | strike slip | 0 | | 136 |
| ☑ | 32.55 | San Jacinto;SJV+A+CC+B | CA | | 90 | V | strike slip | 0.1 | | 170 |
| ☑ | 32.55 | San Jacinto;SJV+A+CC+B+SM | CA | | 90 | V | | 0.1 | | 196 |

**Earthquake Hazards Program**

# 2008 National Seismic Hazard Maps - Source Parameters

New Search

| Fault Name | State |
|---|---|
| Elsinore;GI+T | California |

**MODEL VALUES**

| | |
|---|---|
| Fault parallel slip rate | 5.00 |
| Probability of activity | 1 |

| | ELLSWORTH | HANKS |
|---|---|---|
| Minimum magnitude | 6.5 | 6.5 |
| Maximum magnitude | 7.288 | 7.187 |

**FAULT GEOMETRY**

| | |
|---|---|
| Dip (degrees) | 90 |
| Dip direction | V |
| Sense of slip | strike slip |
| Rupture top (km) | 0 |
| Rupture bottom (km) | 14 |
| Rake (degrees) | 180 |
| Length (km) | 78 |

| Fault Model | Deformation Model | Char Mag[1] | Char Rate[1] | Apriori Rate | Weight |
|---|---|---|---|---|---|
| Moment Balanced | 2.1 | 7.288/7.187 | 1.35e-04/ 1.35e-04 | 9.62e-04 | 0.5 |

[1] 1st Value is based on Ellsworth relation and 2nd value is based on Hanks and Bakun relation

**Earthquake Hazards Program**

# Oops! — File Not Found (404)

**Did you follow a link on our website?**

If you reached this page from another part of earthquake.usgs.gov, please use our Contact Form so we can correct our mistake.

**Did you follow a link from another website?**

Please check our **site map** at the bottom of any of our webpages or Search our website to find the page's new location. Do not send us an email about the broken link because it's not on our site, and we can't fix it.

**Did you type the URL?**

You may have typed the address (URL) incorrectly. Check to make sure you have the correct spelling, capitalization, etc.

# Probabilistic Seismic Hazards Mapping Ground Motion Page

### User Selected Site

| | |
|---|---|
| Longitude | -117.239 |
| Latitude | 33.6005 |

## Ground Motions for User Selected Site

Ground motions (10% probability of being exceeded in 50 years) are expressed as a fraction of the acceleration due to gravity (g). Three values of ground motion are shown, peak ground acceleration (Pga), spectral acceleration(Sa) at short (0.2 second) and moderately long (1.0 second) periods. Ground motion values are also modified by the local site soil conditions. Each ground motion value is shown for 3 different site conditions: firm rock (conditions on the boundary between site categories B and C as defined by the building code), soft rock (site category C) and alluvium (site category D).

| Ground Motion | Firm Rock | Soft Rock | Alluvium |
|---|---|---|---|
| **Pga** | 0.532 | 0.532 | 0.532 |
| **Sa 0.2 sec** | 1.24 | 1.24 | 1.245 |
| **Sa 1.0 sec** | 0.457 | 0.55 | 0.632 |

NEHRP Soil Corrections were used to calculate Soft Rock and Alluvium. *Ground Motion values were interpolated from a grid (0.05 degree spacing) of calculated values. Interpolated ground motion may not equal values calculated for a specific site, therefore these values are not intended for design or analysis.*

# USGS Design Maps Summary Report

## User–Specified Input

**Report Title** Marimina, LLC/Nova Homes, Inc., 35775 Iodine Springs Road, Wildomar, CA

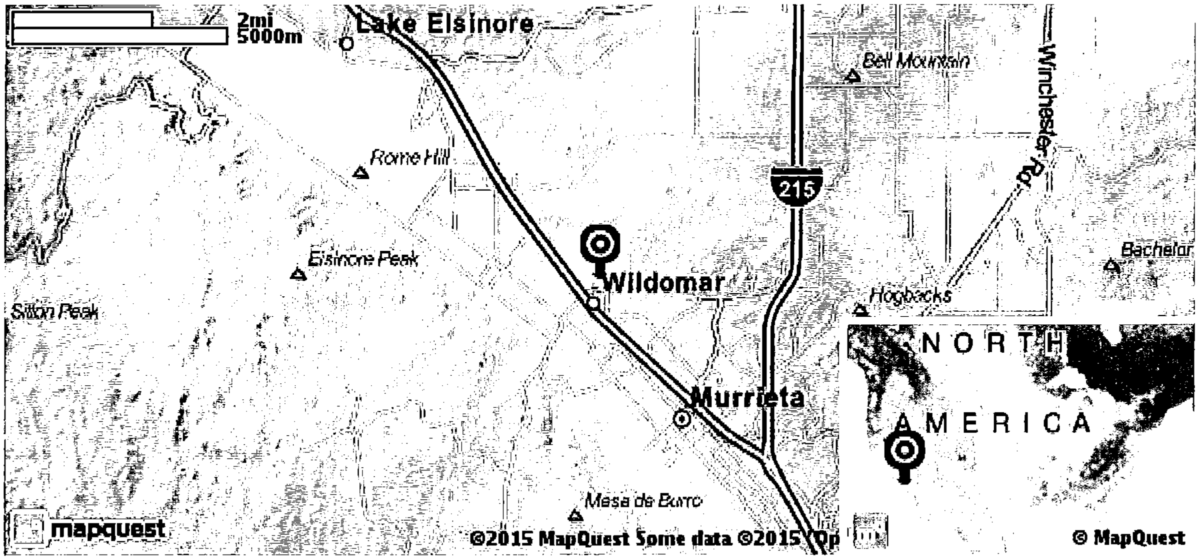
Tue June 23, 2015 21:32:12 UTC

**Building Code Reference Document** ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

**Site Coordinates** 33.60048°N, 117.23897°W

**Site Soil Classification** Site Class C – "Very Dense Soil and Soft Rock"

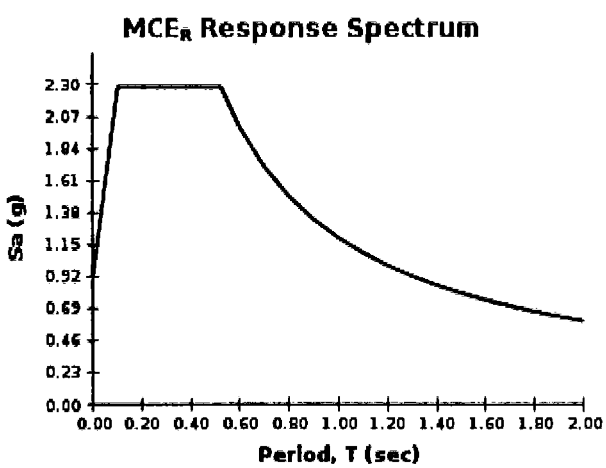
**Risk Category** I/II/III

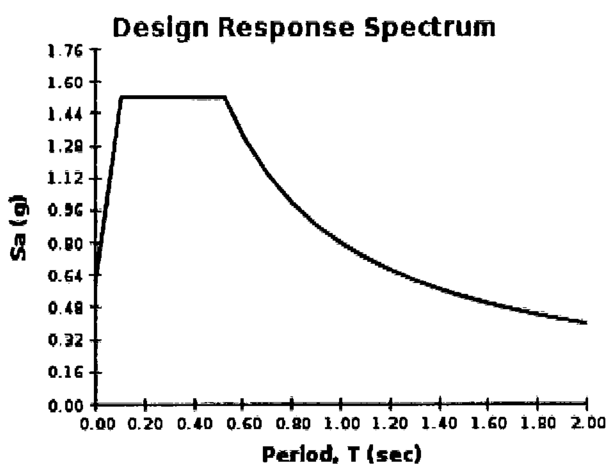



## USGS–Provided Output

| | | | | | |
|---|---|---|---|---|---|
| $S_S$ = | 2.279 g | $S_{MS}$ = | 2.279 g | $S_{DS}$ = | 1.519 g |
| $S_1$ = | 0.921 g | $S_{M1}$ = | 1.198 g | $S_{D1}$ = | 0.799 g |

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.



# USGS Design Maps Detailed Report

## ASCE 7-10 Standard (33.60048°N, 117.23897°W)

Site Class C – "Very Dense Soil and Soft Rock", Risk Category I/II/III

## Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain $S_S$) and 1.3 (to obtain $S_1$). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

**From Figure 22-1** [1] $S_S = 2.279$ g

---

**From Figure 22-2** [2] $S_1 = 0.921$ g

---

## Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class C, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

| **Site Class** | $\bar{v}_s$ | $\bar{N}$ or $\bar{N}_{ch}$ | $\bar{s}_u$ |
|---|---|---|---|
| A. Hard Rock | >5,000 ft/s | N/A | N/A |
| B. Rock | 2,500 to 5,000 ft/s | N/A | N/A |
| C. Very dense soil and soft rock | 1,200 to 2,500 ft/s | >50 | >2,000 psf |
| D. Stiff Soil | 600 to 1,200 ft/s | 15 to 50 | 1,000 to 2,000 psf |
| E. Soft clay soil | <600 ft/s | <15 | <1,000 psf |
| | Any profile with more than 10 ft of soil having the characteristics:<br>• Plasticity index $PI > 20$,<br>• Moisture content $w \geq 40\%$, and<br>• Undrained shear strength $\bar{s}_u < 500$ psf | | |
| F. Soils requiring site response analysis in accordance with Section 21.1 | See Section 20.3.1 | | |

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

## Section 11.4.3 — Site Coefficients and Risk-Targeted Maximum Considered Earthquake ($MCE_R$) Spectral Response Acceleration Parameters

Table 11.4-1: Site Coefficient $F_a$

| Site Class | Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period | | | | |
|---|---|---|---|---|---|
| | $S_S \le 0.25$ | $S_S = 0.50$ | $S_S = 0.75$ | $S_S = 1.00$ | $S_S \ge 1.25$ |
| A | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 |
| B | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| C | 1.2 | 1.2 | 1.1 | 1.0 | 1.0 |
| D | 1.6 | 1.4 | 1.2 | 1.1 | 1.0 |
| E | 2.5 | 1.7 | 1.2 | 0.9 | 0.9 |
| F | See Section 11.4.7 of ASCE 7 | | | | |

Note: Use straight-line interpolation for intermediate values of $S_S$

**For Site Class = C and $S_s$ = 2.279 g, $F_a$ = 1.000**

Table 11.4-2: Site Coefficient $F_v$

| Site Class | Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1-s Period | | | | |
|---|---|---|---|---|---|
| | $S_1 \le 0.10$ | $S_1 = 0.20$ | $S_1 = 0.30$ | $S_1 = 0.40$ | $S_1 \ge 0.50$ |
| A | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 |
| B | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| C | 1.7 | 1.6 | 1.5 | 1.4 | 1.3 |
| D | 2.4 | 2.0 | 1.8 | 1.6 | 1.5 |
| E | 3.5 | 3.2 | 2.8 | 2.4 | 2.4 |
| F | See Section 11.4.7 of ASCE 7 | | | | |

Note: Use straight-line interpolation for intermediate values of $S_1$

**For Site Class = C and $S_1$ = 0.921 g, $F_v$ = 1.300**

**Equation (11.4-1):** $S_{MS} = F_aS_S = 1.000 \times 2.279 = 2.279$ g

**Equation (11.4-2):** $S_{M1} = F_vS_1 = 1.300 \times 0.921 = 1.198$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

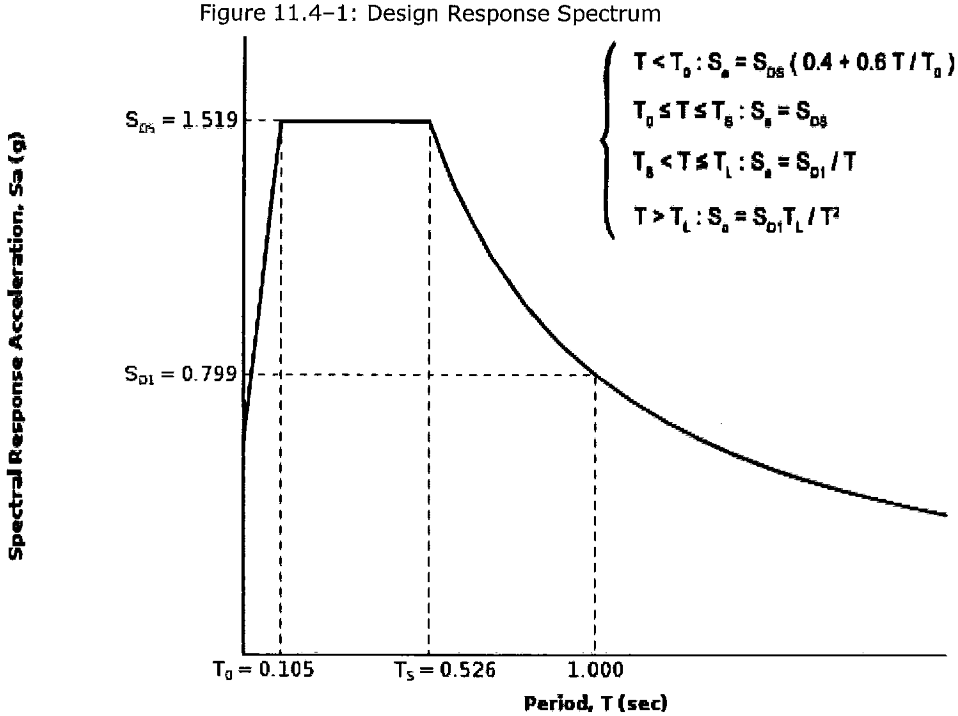
**Equation (11.4-3):** $S_{DS} = ⅔\, S_{MS} = ⅔ \times 2.279 = 1.519$ g

**Equation (11.4-4):** $S_{D1} = ⅔\, S_{M1} = ⅔ \times 1.198 = 0.799$ g

Section 11.4.5 — Design Response Spectrum

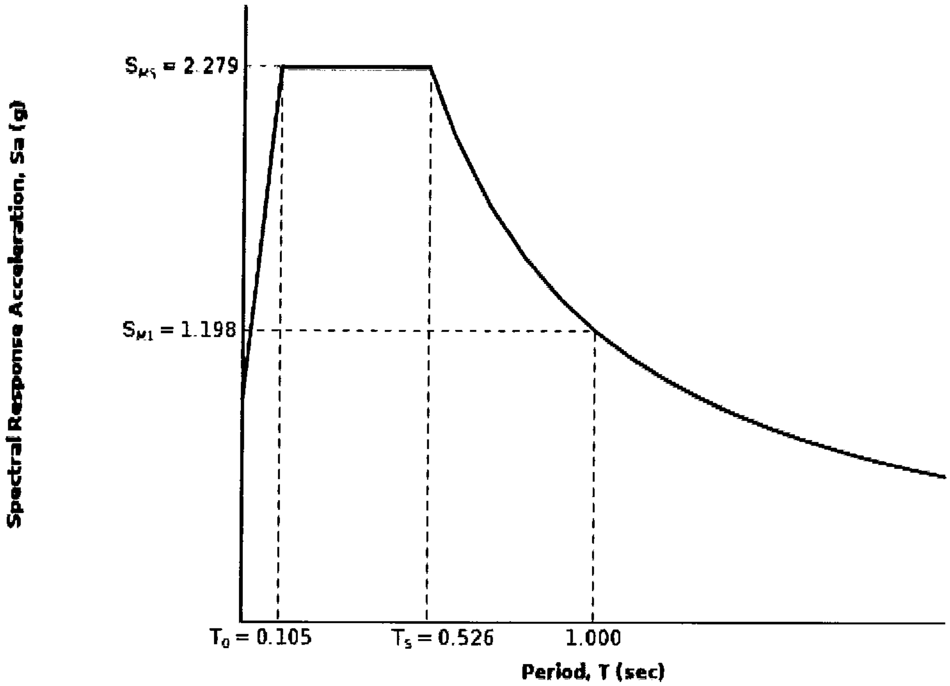
**From Figure 22-12** [3] $T_L$ = 8 seconds

Figure 11.4-1: Design Response Spectrum



## Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake ($MCE_R$) Response Spectrum

The $MCE_R$ Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

**From Figure 22-7** [4] PGA = 0.906

**Equation (11.8–1):** $PGA_M = F_{PGA}PGA = 1.000 \times 0.906 = 0.906$ g

Table 11.8–1: Site Coefficient $F_{PGA}$

| Site Class | Mapped MCE Geometric Mean Peak Ground Acceleration, PGA | | | | |
|---|---|---|---|---|---|
| | PGA ≤ 0.10 | PGA = 0.20 | PGA = 0.30 | PGA = 0.40 | PGA ≥ 0.50 |
| A | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 |
| B | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| C | 1.2 | 1.2 | 1.1 | 1.0 | 1.0 |
| D | 1.6 | 1.4 | 1.2 | 1.1 | 1.0 |
| E | 2.5 | 1.7 | 1.2 | 0.9 | 0.9 |
| F | See Section 11.4.7 of ASCE 7 | | | | |

Note: Use straight–line interpolation for intermediate values of PGA

**For Site Class = C and PGA = 0.906 g, $F_{PGA}$ = 1.000**

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

**From Figure 22-17** [5] $C_{RS}$ = 0.911

**From Figure 22-18** [6] $C_{R1}$ = 0.897

## Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

| VALUE OF $S_{DS}$ | RISK CATEGORY | | |
|---|---|---|---|
| | **I or II** | **III** | **IV** |
| **$S_{DS} < 0.167g$** | A | A | A |
| **$0.167g \le S_{DS} < 0.33g$** | B | B | C |
| **$0.33g \le S_{DS} < 0.50g$** | C | C | D |
| **$0.50g \le S_{DS}$** | D | D | D |

**For Risk Category = I and $S_{DS}$ = 1.519 g, Seismic Design Category = D**

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

| VALUE OF $S_{D1}$ | RISK CATEGORY | | |
|---|---|---|---|
| | **I or II** | **III** | **IV** |
| **$S_{D1} < 0.067g$** | A | A | A |
| **$0.067g \le S_{D1} < 0.133g$** | B | B | C |
| **$0.133g \le S_{D1} < 0.20g$** | C | C | D |
| **$0.20g \le S_{D1}$** | D | D | D |

**For Risk Category = I and $S_{D1}$ = 0.799 g, Seismic Design Category = D**

Note: When $S_1$ is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = E

---

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

## References

1. *Figure 22-1*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

## PROFESSIONAL LIMITATIONS

Our investigation was performed using the degree of care and skill ordinarily exercised, under similar circumstances by other reputable Soils Engineers practicing in these general or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report.

The investigations are based on soil samples only, consequently the recommendations provided shall be considered "preliminary'. The samples taken and used for testing and the observations made are believed representative of site conditions; however, soil and geologic conditions can vary significantly between test excavations. If this occurs, the Project Soils Engineer must evaluate the changed conditions, and designs adjusted as required or alternate design recommended.

The 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 project architect and engineers. Appropriate recommendations should be incorporated into structural plans. The necessary steps should be taken to see that out such recommendations in field.

The findings of this report are valid as of this present date. However, changes in the conditions of a property can occur with the passage of time, whether they due to natural process or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur from legislation or broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by change outside of our control. Therefore, this report is subject to review and should be updated after a period of one year.

## RECOMMENDED SERVICES

The review of grading plans and specifications, field observations and testing by a geotechnical representative of this office is integral part of the conclusions and recommendations made in this report. If Soils Southwest, Inc. (SSW) is not retained for these services, the Client agrees to assume SSW's responsibility for any potential claims that may arise during and after construction, or during the life-time use of the structure and its appurtenants.

The recommendations supplied should be considered valid and applicable, provided the following conditions, in minimum, are met:

i. Pre-grade meeting with contractor, public agency and soils engineer,
ii. Excavated bottom inspections and verification s by soils engineer prior to backfill placement,
iii. Continuous observations and testing during site preparation and structural fill soils placement,
iv. Observation and inspection of footing trenching prior to steel and concrete placement,
v. Subgrade verifications including plumbing trench backfills prior to concrete slab-on-grade placement,
vi. On and off-site utility trench backfill testing and verifications,
vii. Precise-grading plan review, and
viii. Consultations as required during construction, or upon your request.

Soils Southwest, Inc. will assume no responsibility for any structural distresses during its life-time use; in event the above conditions are not strictly fulfilled.