APPENDIX C.
GEOLOGY AND SOILS
BACKGROUND INFORMATION

TECHNICAL REPORT:
GEOTECHNICAL AND COASTAL HAZARDS REVIEW
Loperena Minor Use Permit/Coastal Development Permit
APN 064-253-07, Studio Drive, Cayucos, San Luis Obispo County, California

for
SWCA ENVIRONMENTAL CONSULTANTS
1422 Monterey Street, Suite C200
San Luis Obispo, California 93401

by
COTTON, SHIRES AND ASSOCIATES, INC.
550 St. Charles Drive, Suite 108
Thousand Oaks, California 91360

May 2011

COTTON, SHIRES AND ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

May 31, 2011
SC0099

Ms. Shawna Scott
Planning Program Manager
SWCA ENVIRONMENTAL CONSULTANTS
1422 Monterey Street, Suite C200
San Luis Obispo, California 93401

SUBJECT: **Technical Report**
Geotechnical and Coastal Hazards Review
RE: Loperena Minor Use Permit/Coastal Development Permit
APN 064-253-07, Studio Drive, Cayucos, San Luis Obispo County, CA

Dear Ms. Scott:

In accordance with your authorization, Cotton, Shires and Associates, Inc. (CSA) is pleased to present this technical report containing the findings of our EIR-level geotechnical and coastal hazards review conducted in support of your firm's (SWCA) Environmental Impact Report preparation for the Loperena MUD/CDP project in Cayucos, California. This work was performed by CSA in accordance with our proposal to you dated September 28, 2009, and recommendation for Coastal Hazards Study dated February 5, 2011. The Coastal Hazards Study was performed by David W. Skelly, PE of GeoSoils, Inc. and is appended herein.

In this report, we have peer-reviewed the consultant geotechnical reports for the project, and we review and summarize the geotechnical conditions underlying the subject property, the geologic/geotechnical and coastal hazards, and potential impacts posed to the proposed development. We also identify appropriate mitigation measures where necessary in order to minimize potential impacts.

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We appreciate the opportunity to be of service to SWCA on this project. If you have any questions regarding this report, please feel free to call us.

Sincerely,

COTTON, SHIRES AND ASSOCIATES, INC.

Michael B. Phipps
Principal Engineering Geologist
PG 5748, CEG 1832

Patrick O. Shires
Sr. Principal Geotechnical Engineer
GE 770

MP:DTS:POS:st

TECHNICAL REPORT: GEOTECHNICAL AND COASTAL HAZARDS REVIEW
Loperena Minor Use Permit/Coastal Development Permit
APN 064-253-07, Studio Drive, Cayucos, San Luis Obispo County, California

Table of Contents

		Page
1.0	INTRODUCTION	1
1.1	Project Description	1
1.2	Purpose and Scope of Work	2
1.3	Proposed Development	3
1.4	Vertical Datum	4
1.5	Previous Studies	5
1.6	Peer Review Comments	5
1.6.1	Engineering Geology Comments	5
1.6.2	Geotechnical Engineering Comments	6
2.0	PHYSICAL AND GEOLOGIC SETTING	7
2.1	Terrain	7
2.2	Development History	8
2.3	Geologic Setting	9
2.4	Seismic Setting	10
2.4.1	Deterministic Analysis	11
2.4.2	Probabilistic Analysis	12
3.0	SITE CONDITIONS	12
3.1	Surface Conditions	12
3.2	Subsurface Conditions	13
3.3	Groundwater Conditions	15
3.4	Coastal Bluff Interpretation	15
4.0	HAZARDS REVIEW	18
4.1	Geologic/Geotechnical Hazards	18
4.1.1	Earthquake Fault Rupture	18
4.1.2	Seismic Hazards	18
4.1.3	Expansive Soil	21
4.1.4	Landslides	21
4.1.5	Slope Instability	22
4.1.6	Environmental Hazards from Earth Materials	22
4.1.7	Flooding and Drainage	23

4.2 Coastal Hazards 24
4.2.1 Erosion Hazard 25
4.2.2 Oceanographic Flooding Hazard 26
4.2.3 Breaking Wave Elevation 26
4.2.4 Wave Runup Hazard 26
4.3 Hazards Summary/Conclusions 27

5.0 RECOMMENDATIONS 28
5.1 Update Reports 28
5.2 Site Drainage Plan 28
5.3 DEIR Review 29

6.0 INVESTIGATION LIMITATIONS 29

7.0 REFERENCES 30
7.1 Aerial Photographs 33

APPENDIX A
Discussion of Coastal Hazards and Wave Runup, Northwest and Immediately Adjacent to 2612 Studio Drive (APN 064-253-07), Cayucos, San Luis Obispo County, California (GeoSoils, Inc., 2011).

TABLE

1 Earthquake Faults and Seismicity 11

FIGURES **Follows Page**

1 Regional Geologic Hazard Map 10
2 Regional Fault Location Map 11
3 Engineering Geologic Cross-Section 1-1′ 13
4 Engineering Geologic Cross-Section 2-2′ 13
5 1937 Aerial Photograph Features 17
6 Bluff Edge Delineation on 2005 Aerial Photo 17

PLATES

1 Engineering Geologic Map ..... in pocket

TECHNICAL REPORT:
GEOTECHNICAL AND COASTAL HAZARDS REVIEW

Loperena Minor Use Permit/Coastal Development Permit
APN 064-253-07, Studio Drive, Cayucos, San Luis Obispo County, California

1.0 INTRODUCTION

1.1 Project Description

Cotton, Shires and Associates, Inc. (CSA) is pleased to present the results of our technical review of geologic/geotechnical and coastal hazards, conducted in support of SWCA's preparation of an Environmental Impact Report for the proposed Loperena development project at the north end of Studio Drive in Cayucos, California. The subject property, Assessor's Parcel Number 064-253-07, is located in the unincorporated community of Cayucos, within San Luis Obispo County, California. The Loperena site consists of an irregular-shaped 3,445 square foot parcel of unimproved land located on the northern end of Studio Drive, approximately 250 feet south of the intersection of Studio Drive and California Highway 1. The site is generally bounded by Morro Strand State Beach to the northwest and southwest, an existing residence to the southeast (at 2612 Studio Drive), and the County right-of-way (Studio Drive) to the northeast.

We performed our scope of work in accordance with our proposal to you dated September 28, 2009 (as amended December 1, 2009) and our "Recommendation for Coastal Hazards Study" letter of February 5, 2011. Field reconnaissance and geologic mapping were performed on January 31 and February 4, 2011. CSA was assisted in the Coastal Hazards review by Mr. David Skelly, PE of GeoSoils, Inc., a well-known expert in the field of coastal engineering. Following a review of the existing coastal hazards information, a supplemental coastal hazards study was conducted by Skelly (GeoSoils, Inc., 2011). This study is appended to this report (Appendix A) and summarized in Section 4.2.

1.2 Purpose and Scope of Work

CSA's role on this project was to provide subconsultant services to SWCA in the specialty areas of Engineering Geology, Geotechnical Engineering, and Coastal Engineering/Hazards Assessment. In their preparation of the Initial Study (IS)/Environmental Checklist for the project, the County of San Luis Obispo (2009) identified a number of action items in the Geology and Soils, and Hazards section of the Initial Study that need to be addressed in the project EIR. These included the need to evaluate potential impacts from geologic hazards and site alteration, including the potential hazards of faulting, liquefaction and slope stability, as well as evaluating potentially significant drainage, erosion, and sedimentation impacts from the hazards of wave run-up, storm surge, tsunami, and flooding.

Significant studies addressing geologic hazards, coastal hazards, and geotechnical engineering recommendations for the project development were prepared by the applicant's consultants (Cleath 2006, 2007a, 2007b, 2007c; GSI Soils, Inc., 2007). These documents were peer reviewed by the County of San Luis Obispo and reviewed by other interested parties whose comments are part of the project documentation reviewed herein (County of San Luis Obispo, 2007, 2008, 2009; Haro, Kasunich and Associates, 2007; LandSet Engineers, Inc., 2008. As such, our objectives were to: 1) perform an independent peer review of the project documentation from engineering geologic, geotechnical engineering, and coastal engineering perspectives; 2) independently evaluate the site conditions, based on (and limited to) surficial investigation (i.e., no subsurface exploration was performed by CSA); 3) perform a supplemental coastal hazards study as deemed necessary; and 4) review and summarize geologic and coastal hazards and recommend potential mitigation measures where appropriate.

The specific scope of work performed for this review included the following tasks:

1) Background Data Review: published geologic maps and reports, topographic maps, oceanographic data, survey datum information, aerial photographs, hazard assessments; applicant's materials: reports, data, letters, development plans; project correspondence including peer

reviews and Initial Study by the County of San Luis Obispo and reviews by others;

2) Site areal reconnaissance and surficial geologic mapping to confirm exposed geology and structure, drainage, sedimentation and erosion characteristics, coastal and fluvial bluff locations, and other pertinent features;
3) Data compilation and preparation of geologic map and cross-section;
4) Geologic, Engineering, and Coastal Hazards Review and Analyses; and
5) Preparation of this report.

1.3 Proposed Development

Our understanding of the project is based upon review of the most recent set (4 sheets) of project plans dated January 26, 2011 titled "Jack Loperena Residence, Studio Drive, Cayucos, CA" prepared by Shoreline Engineering and James Maul-Architect. We understand that the proposed development consists of a 2,897 square foot single-family residence including one main floor and a 1,040 square foot basement (which is also the residence footprint). The first floor is designed to be cantilevered out oceanward of the basement footprint and elevated above the beach. An elevated concrete driveway within the County right-of-way would provide access to the residence, sloping down toward the residence from approximate elevation 31.0 at Studio Drive to elevation 26.0 feet at the proposed garage. Retaining walls are required on the south, east and north sides of the residence as well as the north and south sides of the driveway.

Estimated grading quantities include cut of 400 cubic yards, fill of 150 cubic yards (including backfill between retaining walls supporting driveway) resulting in a net export of 250 cubic yards (Shoreline Engineering, 2011). The maximum cut based on the plan will be approximately 12 feet below existing grade along the southern wall of the basement. The maximum fill depth per plan is approximately 7 feet beneath the proposed driveway. Actual cut depth and fill thickness will likely be greater due to construction cuts necessary for foundation construction and unsuitable soils removals required beneath proposed fill areas.

As illustrated by the "Plot Plan" (Shoreline Engineering, 2005), proposed drainage improvements include removal of an existing overside drain, which currently conveys

street runoff from Studio Drive toward the subject property, and construction of a new drop-inlet at the street and 24-inch diameter storm drain which conveys the runoff to an exit structure with energy dissipators north of the northerly driveway retaining wall. An entry apron is proposed to collect drainage south of the southerly driveway retaining wall and tie into a 24-inch diameter storm drain proposed beneath the driveway. The elevated driveway construction in the County right-of-way will require relocation of an 8-inch diameter high pressure natural gas main around the proposed driveway.

1.4 Vertical Datum

We understand the project survey elevations are based upon the NAVD88 Vertical Datum (Volbrecht, undated), and that the topography illustrated on the project plans was surveyed in May 2003 (Shoreline Engineering, personal communication, March, 2011). For the purposes of this report, including the supplemental coastal hazards study by David Skelly, PE (GeoSoils, Inc., 2011) contained in Appendix A, all vertical datum reference is NAVD88 unless otherwise specified. The proposed finish floor elevations of the basement and first floor are 15.0 feet and 26.0 feet NAVD88, respectively. An older "Plot Plan" by Shoreline Engineering dated September 12, 2005 identifies the first floor finish floor elevation as being 28.00 feet; however, we understand this plan is outdated as far as proposed elevations are concerned, and the residence plans govern in this regard (Shoreline Engineering, 2011).

Use of NAVD88 elevations by the applicant's consultants is only problematic when comparing items such as flood elevations indicated on Federal Emergency Management Act (FEMA) Flood Insurance Rate Maps (FIRM) , which for this area are still based on NGVD29, or older regional studies that document wave run-up and tsunami hazards (such as Houston and Garcia, 1978; Kilbourne and Mualchin, 1980). In addition, any survey certifications required for the project by the County of San Luis Obispo may require NGVD29 elevations. Elevations may be converted between the two datums to allow for proper analysis of hazards. This can be done from regional tidal datum elevations (i.e., the nearest tidal data station, Port San Luis, which indicates that NAVD88 minus NGVD29 = 2.92 feet).

1.5 Previous Studies

Five geotechnical (i.e., engineering geologic and geotechnical engineering) reports were prepared by the applicant's consultants (full references provided in the References section):

- Cleath & Associates, May 2, 2006
- Cleath & Associates, January 12, 2007
- GSI Soils, Inc., January 12, 2007
- Cleath & Associates, March 30, 2007
- Cleath & Associates, September 26, 2007

The studies by Cleath included geologic mapping, subsurface exploration, geologic analyses, seismic hazard analyses, identification of geologic hazards and constraints, and recommendations for site development. Cleath (2006, 2007b) also addresses coastal hazards in their reports. The study by GSI Soils, Inc. included subsurface exploration, laboratory testing of soils, and recommendations including, but not limited to, site processing/grading, surface and subsurface drainage, foundations and retaining walls.

1.6 Peer Review Comments

The previous studies referenced in Section 1.5 were peer reviewed from engineering geological and geotechnical engineering perspectives. Based upon that review, we developed the following peer review comments. While some of the comments are design/construction-related items that could be addressed at the building permit stage, we recommend that the applicant's consultants address all of these comments now so that the additional information and recommendations from the consultants can be incorporated into the project EIR.

1.6.1 Engineering Geology Comments – Review of Cleath & Associates (2006, 2007a, b, c) reports generated the following comments:

1. Update seismic design parameters to the current codes (IBC 2009/CBC 2010).
2. Provide reference(s) and information regarding recent activity attributed to the Oceanic Fault.

3. Provide a complete geologic map of the property illustrating the distribution of earth units at the ground surface, including limits of undocumented fill(s) (i.e., more than one generation) and other surficial deposits.
4. Exploration is confined to the north half of the property. There is a lack of subsurface exploration data: 1) near/along the southerly property line where conditions are most critical due to excavations required for basement retaining walls and close proximity of existing development next door; and 2) through the fill slope supporting Studio Drive. Additional subsurface exploration is warranted.
    a. With additional subsurface data, Geologic Cross-Sections A-A′, B-B′ and C-C′ should be extended (or new sections drawn) further into the adjacent property (2612 Studio Drive), with subsurface geology interpreted to at least the property line. Sections should depict the proposed finish floor elevations, proposed retaining walls, and location of anticipated construction cuts relative to adjacent existing structures and road.
    b. Identify the need for special construction measures, such as shoring, slot-cutting, or other methods, as appropriate.
    c. Prepare a longitudinal cross-section extending from Studio Drive to the beach, illustrating the proposed driveway and residence locations, finished floor elevations, and retaining walls. The geometry and extent of existing fill and possible terrace deposits (?), and depth to bedrock beneath the proposed driveway is of most interest.
    d. Recommend special construction measures to protect the road, as appropriate.
5. The cited wave run-up and tsunami elevations are based upon regional studies from 1978-80. A site-specific analysis should be performed to best define the hazard(s).

1.6.2 Geotechnical Engineering Review Comments -- Review of the GSI Soils, Inc. (2007) report generated the following review comments:

1. With a blow count N=7 in SP material at 10′ in B-3 and loose sandy materials described elsewhere, the conclusion that "the shallow bedrock material and the absence of groundwater preclude the potential for liquefaction" is lacking support. Since proposed pier foundations will penetrate these materials (if left in-place) there needs to be a discussion regarding the potential risk of liquefaction at the site, how it will be mitigated, and estimated post mitigation risk levels and distress, taking into account that groundwater can and does rise during wetter months (near the ocean).
2. Analyze dry settlement of loose sand from seismic ground shaking and discuss risks and mitigations.

3. It is unclear if the consultant is recommending removal of all existing fill material (and garbage/deleterious materials) underlying the site. The applicant's consultant should clarify this recommendation.
4. Confirm if construction cuts for removal of existing fill would impact (i.e., cause instability on) Studio Drive or the adjacent property at 2612 Studio Drive.
5. It is unclear if the on-site material is suitable for structural fill. Page 6, section 5.3 item (1) states it's suitable for non-structural fill; then item (3) implies on-site materials are suitable for use as structural fill.
    a. Clarify whether on-site material is suitable for both structural and non-structural fills.
    b. Provide definitions/specifications for structural and non-structural fill.
6. Based on the weak/loose near surface materials, consultant should discuss the suitability of shallow footings for retaining wall foundations.
7. In the discussion regarding temporary excavations and slopes, the consultant indicates that temporary cuts of 1:1 will stand for the short term. Based on the loose materials and depending on whether the fill containing garbage/debris is left in place, 1:1 temporary cuts could potentially be unstable.
8. Based on the proximity of the southern wall of the proposed basement to the property line, it would appear construction cuts may extend onto adjacent developed property.
    a. Clarify whether the adjacent residence, as well as all appurtenant structures and hardscape, would be affected by the proposed construction. Provide appropriate recommendations to mitigate potential distress due to temporary construction excavations.
    b. Provide slope stability analyses for critical temporary (construction) slopes.
9. No bedrock was logged in the borings (except B-3), and bedrock materials were not tested for strength. Define the basis for the geotechnical design parameters for foundations.
10. Boring B-2 had an Expansion Index of 92 at 6 feet. The consultant should discuss this result and determine whether some of the subsurface materials are expansive and if so whether they need to be mitigated.

2.0 PHYSICAL AND GEOLOGIC SETTING

2.1 Terrain

The Loperena property is located in the coastal community of Cayucos in San Luis Obispo County. The general area is characterized by coastal features including beach front adjacent to relatively low coastal and fluvial bluffs that range in elevation from

approximately 30 to 50 feet. Nearby foothills northeast of Highway 1 (aka, Cabrillo Highway) rise moderately to steeply to elevations ranging from 300 to 500 feet. The Loperena property is unique in that it is situated adjacent to the broad mouth and alluvial valley of Old Creek and the property appears to physically sit atop and/or straddle a bedrock remnant of the fluvial bluff that is now mostly buried by artificial fill materials. Topography at the site ranges from approximate elevation 10 feet on the active beach to elevation 26 along the southerly property line. Above the beach, a bedrock outcropping extends to approximate elevation 17 feet, where it is capped by soils, and slopes generally west to northwest at roughly a 2:1 gradient. The remainder of the property slopes northwest at gradients ranging from 2½:1 to 5:1. Within the County right-of-way, an approximately 10-foot high 2:1 gradient fill slope descends west-southwest from the pavement of Studio Drive toward the east property line.

2.2 Development History

A series of aerial photographs, including vertical stereopairs and oblique photos covering the time period from 1937 through 2010 were reviewed. A complete list of aerial photographs reviewed is included in References Section 7.1. This review allowed us to develop a better understanding of fluvial and coastal bluff locations, as well as the pertinent development history of the site and immediate area.

In 1937, Cabrillo Highway was a primitive road aligned landward of its present location, along what is now Ocean Boulevard and Cabrillo Avenue. Studio Drive occupied its present location parallel to the coastline but did not exist in front of the present Loperena property; rather, it returned northeast back to Cabrillo Highway approximately 200 feet south of the present Loperena property frontage. Development in the area was very sparse. A northwest-, west-, and southwest-facing rock outcrop occupied the elevated portion of the Loperena site, descending north to the slightly elevated alluvial plain of Old Creek, and descending west-southwest to the beach above the tidal zone. The northwest-facing portion of the outcrop faced the mouth of Old Creek, while the southwest portion faced the ocean as it does today. The lowland area immediately north of the Loperena property appeared to contain alluvial sediments in the broad valley of Old Creek. The area between the Loperena property and the active creek channel (in 1937) and inland of the active beach, contained a low, broad, slightly vegetated shore-parallel ridge (dune). By 1949, Cabrillo Highway had been realigned

slightly oceanward within the Old Creek drainage, including a new bridge over Old Creek. By 1959, most of the lots on the oceanward side of Studio Drive were developed. Construction on Whale Rock Reservoir had commenced inland on Old Creek, and was reportedly completed in April, 1961 (City of San Luis Obispo, 1998).

The 1963 aerial photographs document major changes, including realignment and widening of Cabrillo Highway oceanward toward Studio Drive, and extension of Studio Drive approximately 450 feet northwest where it then returned back to the main highway. This construction resulted in significant fills being placed across the aforementioned rock outcropping to construct the Studio Drive extension, and significant fills built across the alluvium in the valley of Old Creek to support the highway. The northerly five oceanward properties on Studio Drive, including the Loperena property and those at 2612 through 2618 Studio Drive, south of the Loperena property, were undeveloped at this time. By 1972, the properties at 2614 through 2618 Studio Drive were developed. A dirt parking lot had been graded south of and adjacent to the active channel of Old Creek, near the beach. Development of the property immediately south of the project site, at 2612 Studio Drive, occurred sometime between 1979 and 1986 based on aerial photos for those years. Some fill may have been pushed north onto the project site during grading/construction (pre-1986) of the adjacent properties to the south.

2.3 Geologic Setting

The Loperena site is located on an active beach and adjacent terrace at the edge of the Pacific Ocean at Estero Bay just north of Los Osos Valley. The elevated portion of the site sits atop or slightly straddles the buried edge of a fluvial bluff on the south side of the mouth of the Old Creek drainage. Elevations at the site, including the County right-of-way up to Studio Drive, range from slightly less than 10 feet to 30 feet above present sea level (NAVD88).

The site is located in the Southern Coast Ranges Geomorphic Province. This province is bounded on the east by the San Andreas Fault; on the south by the Santa Ynez Mountains (Western Transverse Ranges Geomorphic Province); on the west by the Continental Borderland offshore; and on the north by the Northern Coast Ranges. Lettis (2004) has defined the southern region of the Southern Coast Ranges along the

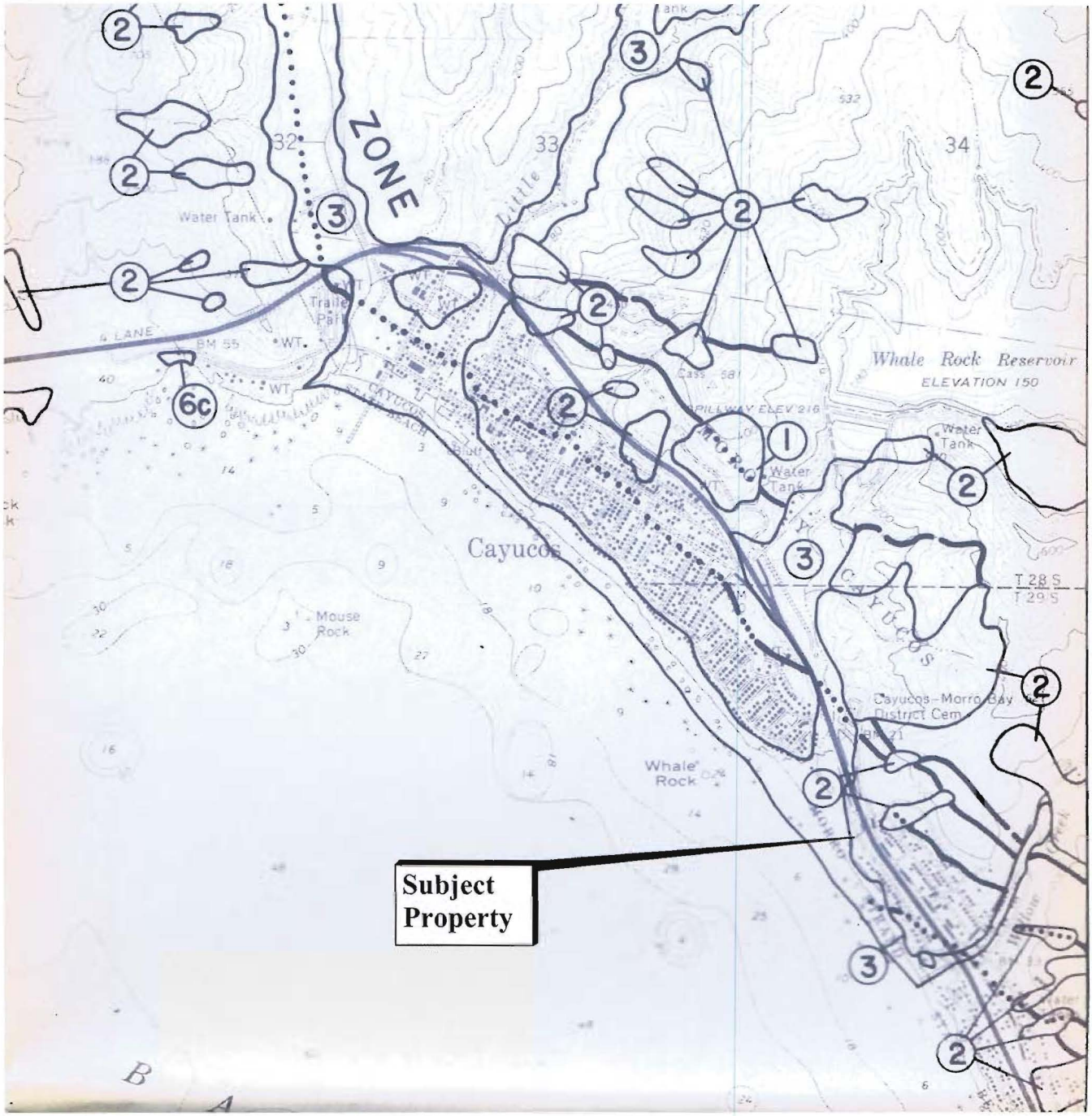
coast (which includes the site) as the Los Osos Domain. This domain is characterized by west-northwest to north-northwest trending mountain ranges and valleys with parallel fault systems bounded on the south by the Santa Ynez River Fault (Western Transverse Ranges); on the north and east by the Oceanic-West Huasna Fault Zone (Santa Lucia Mountains and San Rafael Mountains of the Southen Coast Ranges); and on the west by the Hosgri Fault Zone (Offshore Santa Maria Domain within the Continental Borderland). The mountains and valleys in the area include the Santa Lucia Mountains, Los Osos Valley, and San Luis Mountains.

The area is characterized by west-northwest trending reverse faults and tight subparallel folding of rocks assigned to Franciscan Melange (generally greywacke, shale, greenstone, and serpentine). These rocks are typically chaotically fractured. Other geologic units in the area include Coast Range Ophiolite (Serpentine) found within the Franciscan Complex and Mesozoic Great Valley Sequence. Quaternary marine terrace and older alluvium deposits overlie these rocks along the coast in the vicinity of the site. These units are relatively thin where mapped at the subject site, and likely represent the last sea-level highstand (wave-cut platform at the base of the marine terrace deposits).

Landslides are present within the Franciscan Complex rocks in the site area (Figure 1), including a massive deep-seated ancient landslide located approximately 2,000 feet up-canyon from the site, along Old Creek near the Cayucos Morro Bay Cemetery, and a large earthflow landslide, the toe of which occurs approximately 400 feet northeast of the site across Highway 1.

2.4 Seismic Setting

The subject site is located within an area of high seismicity. The nearest and controlling faults, with respect to site ground shaking, are the Hosgri Fault, located approximately 8.1 miles west of the site; the Los Osos Fault, located approximately 11.1 miles south of the site; and the San Luis Range Fault, located approximately 14 miles east of the site. There are several northwest trending, parallel fault systems in the vicinity of the subject site (including the San Andreas Fault). The most proximal fault is Cambria Fault, which is mapped within 1 kilometer of the site and the trend of which projects near the site (Lettis, 2004). Other faults near the site are the Hosgri-San Simeon Fault Zone, Los Osos



EXPLANATION

1	Young Landslides (Aerial photo and field examinations)
2	Quaternary Landslides (Hall, 1974; Hall and Prior, 1975)
3	Unconsolidated alluvium (Hall and Prior, 1975). These are areas where a shallow groundwater table probably exists, and therefore have the potential for seismically induced liquefaction
4	Faults: solid line where well located, dashed line where inferred, dotted line where concealed (Hall and Prior, 1975; Hall, 1974; Jennings, 1977; Wagner, 1974)
6c	Unique geologic features of scientific and/or educational value

N

0 2000 ft 4000 ft

1"= 2000'

Reference: Kilbourne and Sholin, 1980, CDMG, OFR 80-6, Plate 1a, Geology for Planning, Cayucos 7.5 Minute Quadrangle

COTTON, SHIRES AND ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

REGIONAL GEOLOGIC HAZARD MAP
Loperena Property
APN 064-253-07, Studio Drive
CAYUCOS, CALIFORNIA

GEO/ENG BY	SCALE	PROJECT NO.
SS	1" = 2000'	SC0099
APPROVED BY	DATE	FIGURE NO.
MP	May 2011	1

Fault Zone, Edna Fault, San Miguelito Fault, Oceanic-West Huasna Fault, East Huasna Fault, and Rinconada Fault. The M6.5 December 22, 2003 San Simeon earthquake apparently caused a peak ground acceleration of approximately 0.16g at the site (California Integrated Seismic Network). The Regional Fault Location Map (Figure 2) depicts the site location relative to the aforementioned faults.

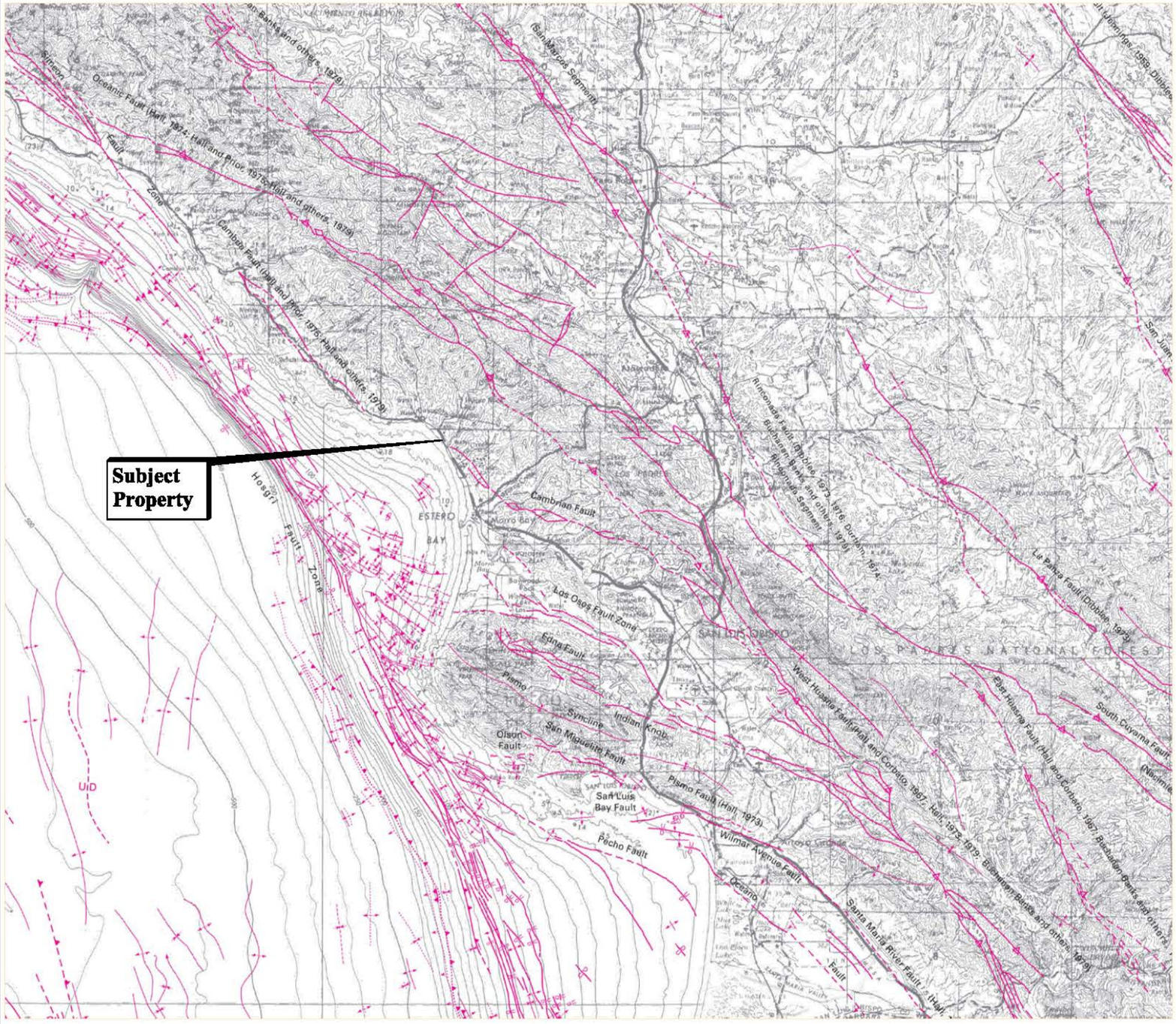
Cleath (2006) states that the Oceanic fault is considered the nearest active fault to the site at 6 km (3.7 miles) distance due to "recent activity attributed to the fault"; however, the report doesn't specify the nature of that activity, nor cite references. Cleath performed a Probabilistic Seismic Hazard Analysis (PSHA) to determine design-basis earthquake parameters, and determined that the design-basis ground motion of the site is 0.29g. Considering the 5-year age of the Cleath (2006) report and multiple references to the 1997 UBC requirements, which have been superseded by several codes up to the 2009 CBC/2010 IBC, the seismic ground motion information and code-required design parameters provided by Cleath should be updated as part of an update report/plan review, as noted in Section 1.6.1 (Engineering Geology Comments) of this report.

CSA performed the following deterministic and probabilistic analyses for comparison purposes:

2.4.1 Deterministic Analysis - The following table provides the results of our deterministic analysis and lists the major earthquake sources, the distances from the sources to the site, the maximum Moment Magnitudes and the Peak Horizontal Ground Accelerations that are anticipated at the site.

Fault Source	Distance (mi/km)	Moment Magnitude[1]	Peak Horizontal Accelerations (g)[2]
Hosgri	8.0/12.8	7.5	0.352
Los Osos	9.1/14.6	7.0	0.348
San Luis Range	14.9/23.9	7.2	0.240

[1]Based on "Probabilistic Seismic Hazard Assessment for the State of California" by CDMG, DMG Open-File Report 96-08.



EXPLANATION

OFFSHORE REGION*

U/D — Active or potentially active high angle fault (sea-floor projection of fault tip where blind or buried)—Deforms early/late Pliocene (2.8–3.4 Ma) unconformity or younger deposits or surfaces; U/D (Up/Down) indicates relative sense of displacement, bar indicates dip direction; dashed where approximately located

Active or potentially active low angle fault (sea-floor projection of fault tip or leading edge of ramp where blind or buried)—Deforms early/late Pliocene (2.8–3.4 Ma) unconformity or younger deposits or surfaces; teeth indicate dip direction; dashed where approximately locate

Active or potentially active anticline axial trace (sea-floor projection where buried)—Arrow indicates direction of plunge; dashed where approximately located

Active or potentially active syncline axial trace (sea-floor projection where buried)—Arrow indicates direction of plunge; dashed where approximately located

Active or potentially active monocline axial trace (sea-floor projection where buried)—Arrow indicates direction of plunge; dashed where approximately located

Inactive fault (bold) or fold (light)—Does not deform early/late Pliocene (2.8–3.4 Ma) unconformity; where this unconformity and (or) younger sediments are absent as a result of erosion, structures are mapped as potentially active

ONSHORE REGION*

Active fault trace—Deforms deposits or surfaces ≤500,000 ka; dashed where approximately located

Potentially active fault trace—May deform deposits or surfaces ≤500,000 ka; dashed where approximately located

Inactive active fault trace—Does not deform deposits or surfaces ≤500,000 ka; dashed where approximately located

Anticline axial trace—Arrow indicates direction of plunge; solid where active or potentially active; dotted where inactive

Syncline axial trace—Arrow indicates direction of plunge; solid where active or potentially active; dotted where inactive

Monocline axial trace—Solid where active or potentially active; dotted where inactive

*Note: See text for discussion of mapping techniques and age criteria used to identify fault activity.

0 6 mi 12 mi
1" = 6 mi

Reference: Lettis, 2004, USGS, Scientific Investigations Map 2812, Bulletin 1995-AA

COTTON, SHIRES AND ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

REGIONAL FAULT LOCATION MAP
Loperena Property
APN 064-253-07, Studio Drive
CAYUCOS, CALIFORNIA

GEO/ENG BY	SCALE	PROJECT NO.
SS	Approx 1" = 6 mi	SC0099
APPROVED BY	DATE	FIGURE NO.
MP	May 2011	2

[2]Based on attenuation relationships developed by Abrahamson and Silva, 1997, Horizontal - Rock as determined using the computer program EQFAULT bv T.F. Blake (1989, and updated 2004).

2.4.2 **Probablistic Analysis** - We performed a probabilistic analysis employing the computer program FRISKSP (by T.F. Blake, 1988 and updated 2004) and incorporated moment magnitudes from the California Division of Mines and Geology (CDMG) publication "Probabilistic Seismic Hazard Assessment For The State of California" (DMG Open File Report 96-08) and attenuation relationships by Abrahamson and Silva (1997 – Horizontal Rock). The results of our probabilistic analysis indicate an appropriate Design Basis Earthquake (10 percent probability of exceedance in 50 years, or a 475 year return interval, which is generally used for residential and commercial buildings) peak horizontal ground acceleration of **0.24g**. We note this is lower than the 0.29g value reported by Cleath (2006), possibly due to differences in attenuation relationships used.

Taking into account the above earthquake Moment Magnitudes and the results of the deterministic and probabilistic approaches, it is our opinion that project area could experience a peak horizontal ground acceleration (PGA) as high as **0.35g** (equal to the deterministic value calculated for an earthquake on the Hosgri Fault for the site).

3.0 SITE CONDITIONS

3.1 Surface Conditions

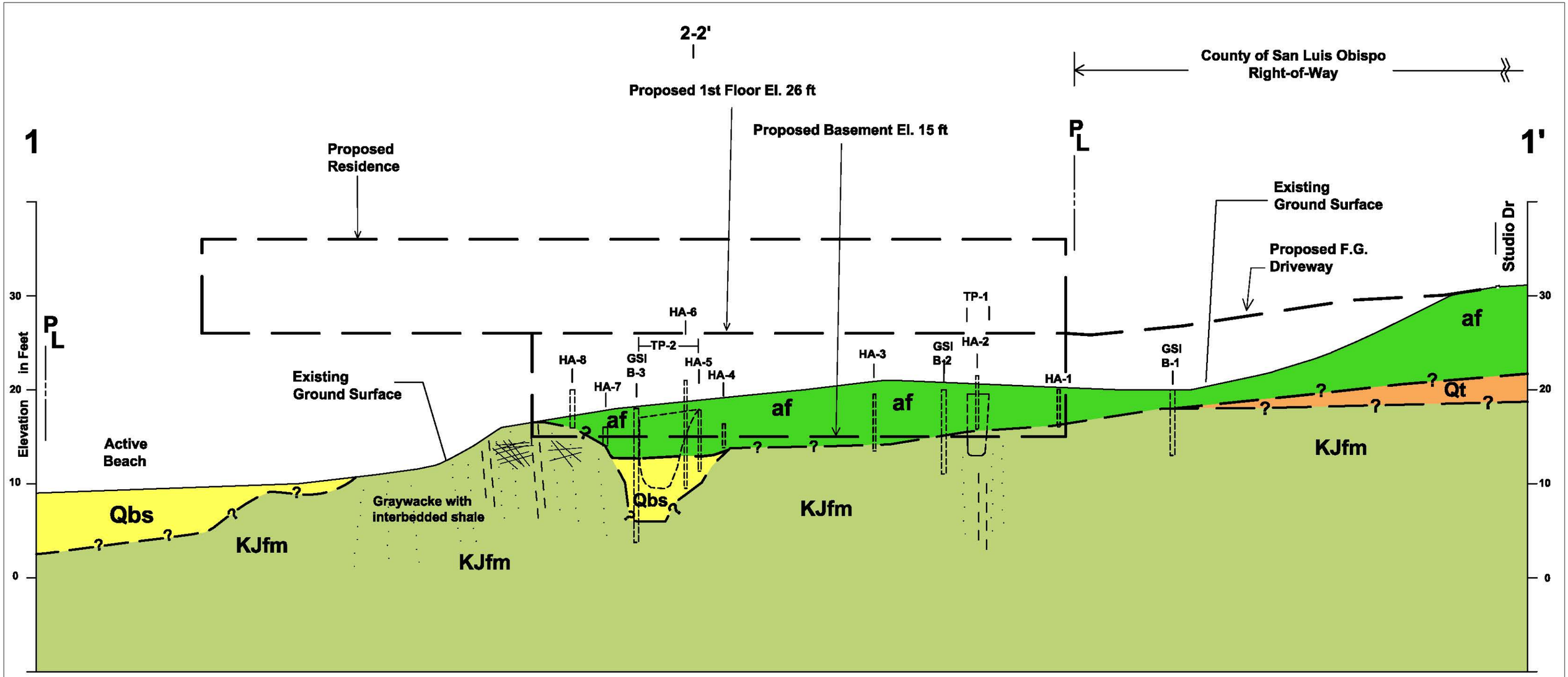
The Loperena property is bounded by Studio Drive and the county right-of-way on the east, an existing residence (2612 Studio Drive) to the south, and Morro Strand State Beach to the north and west. An engineering geologic map (Plate 1) was prepared based upon our site observations and document review, and illustrates the distribution of earth materials exposed at the ground surface. The western portion of the property contains beach sand. A bedrock outcropping consisting of graywacke sandstone with minor shale interbeds is exposed between approximate elevations 10 feet and 17 feet in the center of the property. The greywacke sandstone bedrock is moderately weathered, hard to very hard, and closely fractured. The thin shale beds are soft, intensely fractured, and eroded out (forming indentations in the outcrop) relative to the adjacent

resistant sandstone. Numerous joints and joint sets were mapped with joint spacings as narrow as 4 inches. Relict bedding planes mapped within the bedrock have strikes ranging from N58°W to N75°W and dips ranging from 85° NE to vertical. These orientations are generally consistent with the findings of Cleath (2006). Landward of the bedrock outcropping, the site is covered by an apron of undocumented fill that is covered with extensive iceplant growth. The fill deposits appear to thin immediately north of the site, where they cap older alluvium sediments and possibly dune sediments. A fill slope ascends from the east property line up to the pavement of Studio Drive. Our interpretation of the distribution of earth materials at the site is illustrated on Plate 1, Geologic Map. Two cross-sections (Figures 3 and 4) were prepared to illustrate our interpretation of subsurface conditions based upon our geologic mapping and review of the applicant's consultant's exploration data.

As illustrated in Cleath's Figure 1 (2006) a narrow, natural drainage swale conveys runoff discharging from the existing concrete overside drain from Studio Drive. The overside drain collects drainage from the oceanward side of Studio Drive. This drainage swale trends around the northeast corner and descends toward the beach immediately north of the Loperena property. Erosion in the swale appears to have been accelerated by foot traffic from people accessing the beach. A thin veneer of fill appears to cap older alluvial sediments in this area, based on observations of soils exposed in the swale. North of the swale, the older alluvial sediments may be overlain by thin dune deposits (the low shore-parallel ridge described in the aerial photograph review). Drainage on the landward side of Studio Drive is collected in a concrete drainage ditch located between the oceanward shoulder of Highway 1 and Studio Drive. The drainage ditch trends northwest and ties into a concrete pipe that runs beneath Studio Drive, outletting on the backbeach area north of the Loperena property.

3.2 Subsurface Conditions

Subsurface conditions were explored by the applicant's geotechnical consultants. The study by Cleath (2006) consisted of eight hand-auger borings excavated to depths ranging from 2 to 11.5 feet below ground surface. The study by GSI Soils, Inc. (2007) included subsurface exploration consisting of three small-diameter borings excavated to depths ranging from 7 to 14.25 feet below ground surface. GSI Soils also excavated two exploratory trenches to depths of 7 and 10 feet. Logs for these trenches were provided



N58E

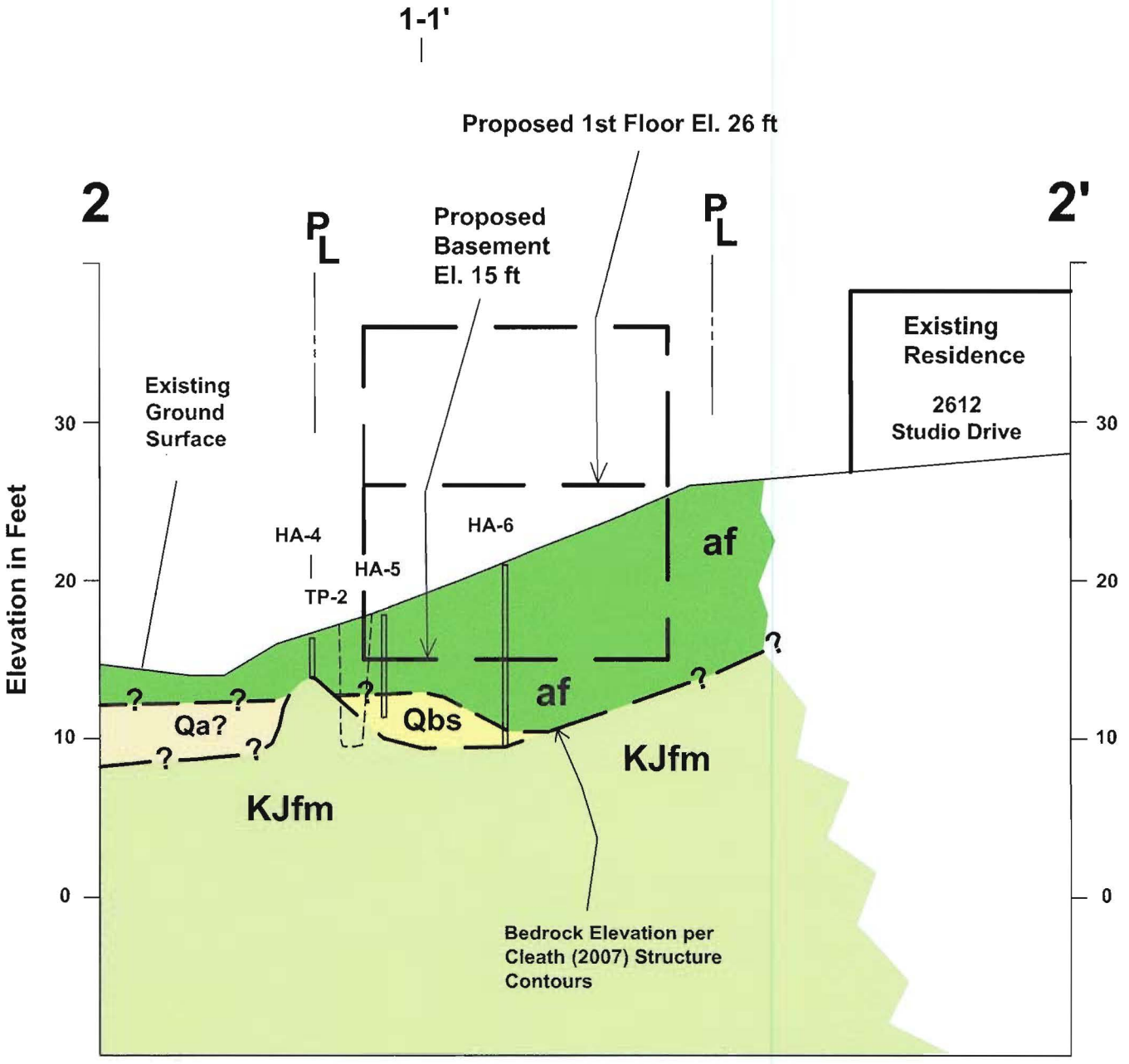
0 5 10
Scale in Feet

See Plate 1 for Explanation

COTTON, SHIRES & ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

Engineering Geologic Cross Section 1-1'
Loperena Property
APN 064-253-07, Studio Drive
CAYUCOS, CALIFORNIA

GEO/ENG BY	SCALE	PROJECT NO.
SS	1 inch = 10 ft	SC0099
APPROVED BY	DATE	FIGURE NO.
MP	May, 2011	3



See Plate 1 for Explanation

N35W

0 5 10
Scale in Feet

COTTON, SHIRES & ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

Engineering Geologic Cross Section 2-2'
Loperena Property
APN 064-253-07, Studio Drive
CAYUCOS, CALIFORNIA

GEO/ENG BY	SCALE	PROJECT NO.
SS	1 inch = 10 ft	SC0099
APPROVED BY	DATE	FIGURE NO.
MP	May, 2011	4

by both Cleath (2007a) and GSI (2007). Based upon the trench observations, Cleath (2007a) revised logs of four of their previous hand-auger borings (HA-1, HA-2, HA-7 and HA-8) in an addendum report. All of the exploration was performed on the subject property with the exception of GSI B-1 which was excavated in the County right-of-way of Studio Drive. One limitation we see in the reviewed data is that the site exploration is confined to the northern half of the property, between the east property line and the bedrock outcrop; hence, there is a lack of subsurface data along or near the southerly property line where the deepest cuts are proposed. This would not normally be an issue due to the narrow,25–foot width of the parcel in the proposed area of development; however, as suggested by Cross-Section 2-2′ (Figure 4), some critical conditions warrant further subsurface exploration, including the maximum retaining wall height, and potentially deep construction cuts along the southerly property line adjacent to an existing residential structure as recommended in Section 1.6 (Peer Review Comments) of this report.

As shown in Cross-Section 1-1′ (Figure 3), the exploration by the project consultants indicates that much of the landward portion of the site is underlain by one or two generations of undocumented artificial fill deposits in the near surface. These materials ranged in thickness from 4.5 to 10.5 feet and consisted of sandy clays, clayey sands and silty sands in a generally loose condition. Debris consisting of trash, plastic, woodchips, and roots was common in the upper 3-5 feet. Loose beach sand was encountered beneath the fill in Cleath HA-5, HA-6, Cleath/GSI TP-2, and GSI B-3 in what appears to be a narrow remnant "cove" that is open to the north. A horizon of dense clayey sand to very stiff sandy clay was encountered beneath the fill in GSI B-1 and B-2. Based on Cleath's log of nearby TP-1, the sandy clay horizon in B-2 is likely weathered mudstone bedrock. The clayey sand horizon in B-1 may be either a terrace deposit or weathered bedrock. Bedrock consisting of hard/indurated greywacke sandstone with thin soft shale interbeds underlies the undocumented fill and possible terrace deposits, but was barely penetrated by the subsurface exploration. Other than what can be deduced from the aerial photograph review, soil and rock conditions underlying Studio Drive fronting the Loperena property are unknown.

3.3 Groundwater Conditions

Groundwater is not described in any of the subsurface exploration logs by Cleath (2006, 2007a) or GSI (2007). Cleath (2006, p.6) concludes that no seepage is evident on the property. GSI also notes a lack of free groundwater in the exploration, but states that perched water conditions in the upper 5 feet should be anticipated during wet winter months above the dense bedrock materials. We anticipate that groundwater is present in the beach sand at or very slightly above sea level. Groundwater may also occur within terrace deposits, which are mapped capping bedrock along Studio Drive on a regional geologic map (Hall, et al, 1975), but appear to pinch out at or near the site. As such, terrace deposits may or may not underlie the southerly portion of the Loperena property or the fill slope supporting Studio Drive in the county right-of-way (both of which were not explored).

Fluctuations in groundwater levels typically occur from variations in rainfall, irrigation, flooding and other factors, and groundwater levels may be different at other times and locations than the exploration performed at the subject site. The most critical groundwater concerns for the project include potential perched groundwater within surficial soils capping the bedrock, and the potential for encountering groundwater in drilled shafts for the proposed pier foundations during construction.

3.4 Coastal Bluff Interpretation

The Initial Study for the project (County of San Luis Obispo, 2009) indicated the need to determine the site's location with respect to the coastal bluff and to provide a recommendation for development bluff setbacks if applicable. Cleath (2006) opined, on the basis of site mapping and review of aerial photographs dating back to 1949, that the landward portion of the property is bounded by a fluvial bluff line at the mouth of Old Creek. Based upon our review of the available data and a sequence of aerial photographs dating back to 1937, from a geological perspective we find that the landward portion of the site does in fact sit atop or slightly straddle a bedrock remnant of a fluvial bluff that is now mostly buried by artificial fill materials. As noted above in Section 2.1 Terrain, 1937 aerial photographs show a northwest-, west-, and southwest-facing rock outcrop occupied the elevated portion of the Loperena site, descending north to the slightly elevated alluvial plain of Old Creek, and descending west-

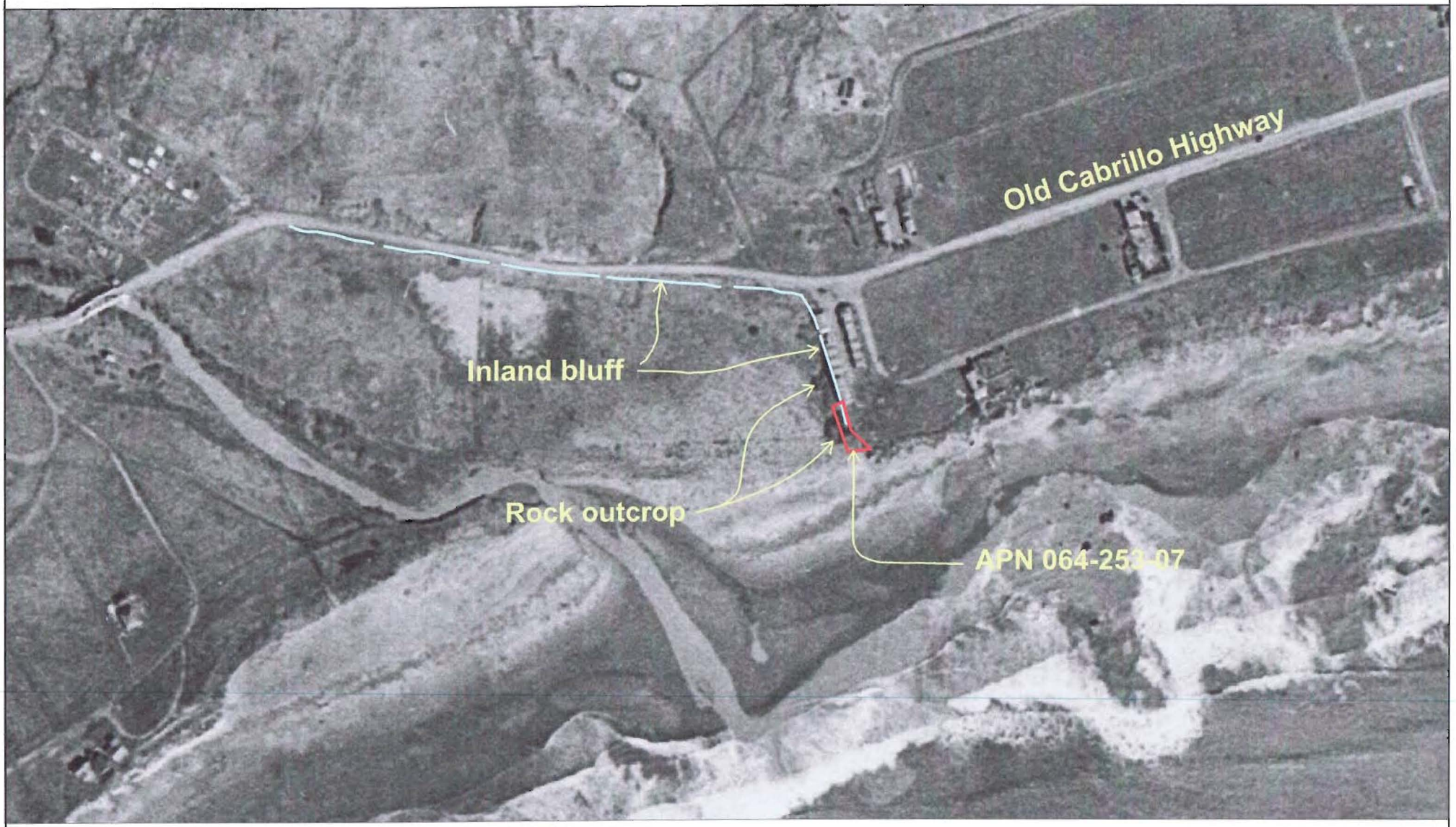
southwest to the beach above the tidal zone. It is important to note the northwest-facing portion of the rock outcrop is approximately perpendicular to the general trend (approx. N40°W) of the shoreline at the mouth of Old Creek. This outcropping extended inland approximately 300 feet (beneath the present alignment of Highway 1), before turning to an approximate N15°W trend (Figure 5); this feature represents the northerly edge of a wave-cut platform that is present throughout Cayucos, including both sides of the Old Creek drainage. The platform would continue north, were it not for the presence of Old Creek meeting the ocean at this location. As such it is reasonable to conclude this portion of the outcropping was formed by fluvial erosion processes (and possibly mass-wasting processes) from the ancestral flow of Old Creek at a time when the creek was entrenched along the southerly side of the creek valley. Evidence for southerly entrenchment in the creek valley includes the massive ancient landslide 2,000 feet up-canyon that displaced the creek approximately 400 feet west—hence the southerly entrenched creek likely removed lateral support in the paleocanyon of Old Creek (i.e., during the last late-Pleistocene glacial stage when eustatic sea level was lower), triggering the landslide. We therefore consider the top of the 300-foot long outcropping, which is oriented perpendicular to the shoreline, to be an inland bluff in the geomorphic sense.

The site topography and aerial photgraphs indicate that the oceanward remainder of the rock outcropping gradually curves to face west and ultimately slightly southwest at the southerly property line of the Loperena property. We consider that the west- to southwest-facing portion of the rock outcropping, which is at about a 45-degree angle to the active shoreline, represents a transition between fluvial bluff-forming/erosion processes and coastal bluff-forming/erosion processes. Along this segment, fluvial processes and possibly mass-wasting processes, were more influential in the geologic past, when the active channel of Old Creek was entrenched on the southern side of the valley, and/or when the creek was topographically lower during a lower stand of eustatic sea level. Coastal erosion processes are more prevalent today, as it is clear that wave action does reach the outcropping in storm surf conditions. This "transition" section of the rock outcropping extends south of the Loperena property approximately 100 feet, to a point on the property at 2614 Studio Drive. Beyond this point the landform generally trends about S47°E and appears wholly influenced by coastal erosion processes and represents true "coastal" bluff in the geomorphic sense.

Cleath (2006) cites the California Code of Regulations and California Coastal Commission (CCC) guidelines for criteria establishing the terminus of a coastal bluff, and presents an analysis implementing these criteria in his Figure 10. We obtained the same information from the CCC (M. Johnsson, 2011) and we understand that CCR Title 14, Section 13577 (h)(2) is the only part of the Coastal Act that defines a bluff edge. The last part of this code section deals with termination of a coastal bluff line versus a canyon or inland bluff line. Specifically, the section states:

> *"The termini of the bluff line, or edge along the seaward face of the bluff, shall be defined as a point reached by bisecting the angle formed by a line coinciding with the general trend of the bluff line along the seaward face of the bluff, and a line coinciding with the general trend of the bluff line along the inland facing portion of the bluff. Five hundred feet shall be the minimum length of bluff line or edge to be used in making these determinations."*

We understand that the five hundred feet rule at the end of the paragraph was inserted to ensure that a reasonable length of bluff was used to differentiate between a coastal bluff and an inland facing bluff (M. Johnsson, 2011). The difficulty in applying these criteria to the Loperena site rests with establishing the general trend of the fluvial/inland bluff along a distance of 500 feet. As noted above, the northwest-facing portion of the rock outcropping is seen in the 1937 photograph extending at least 300 feet inland from its oceanward end on the Loperena property, along a trend of approximately N50°E which is perpendicular to the shoreline. Beyond this point the inland bluff turns to an approximate N15°W trend following what is now Cabrillo Avenue (see Figures 5 and 6). We note that our interpretation of the inland/fluvial bluff line is further east and inland than that illustrated by Cleath (2006, Figure 10) on a 2003 aerial photograph, as it appears Cleath did not consider the position of the bluff prior to it being obscured/buried by the placement of one or more prisms of artificial fill across the creek valley, for the two subsequent realignments of Highway 1 seen in the 1949 and 1963 aerial photographs. However, in our opinion, any reasonable interpretation of a "general trend" for the inland bluff, following the Coastal Commission's guidelines--whether it be the aforementioned 300-foot segment from the oceanward tip of the rock outcropping, or an average trend of the first 500 lineal feet extending inland from the oceanward tip of the rock outcropping--will all result in a determination of the coastal bluff terminus being located southeast of the Loperena property (Figure 6). In this



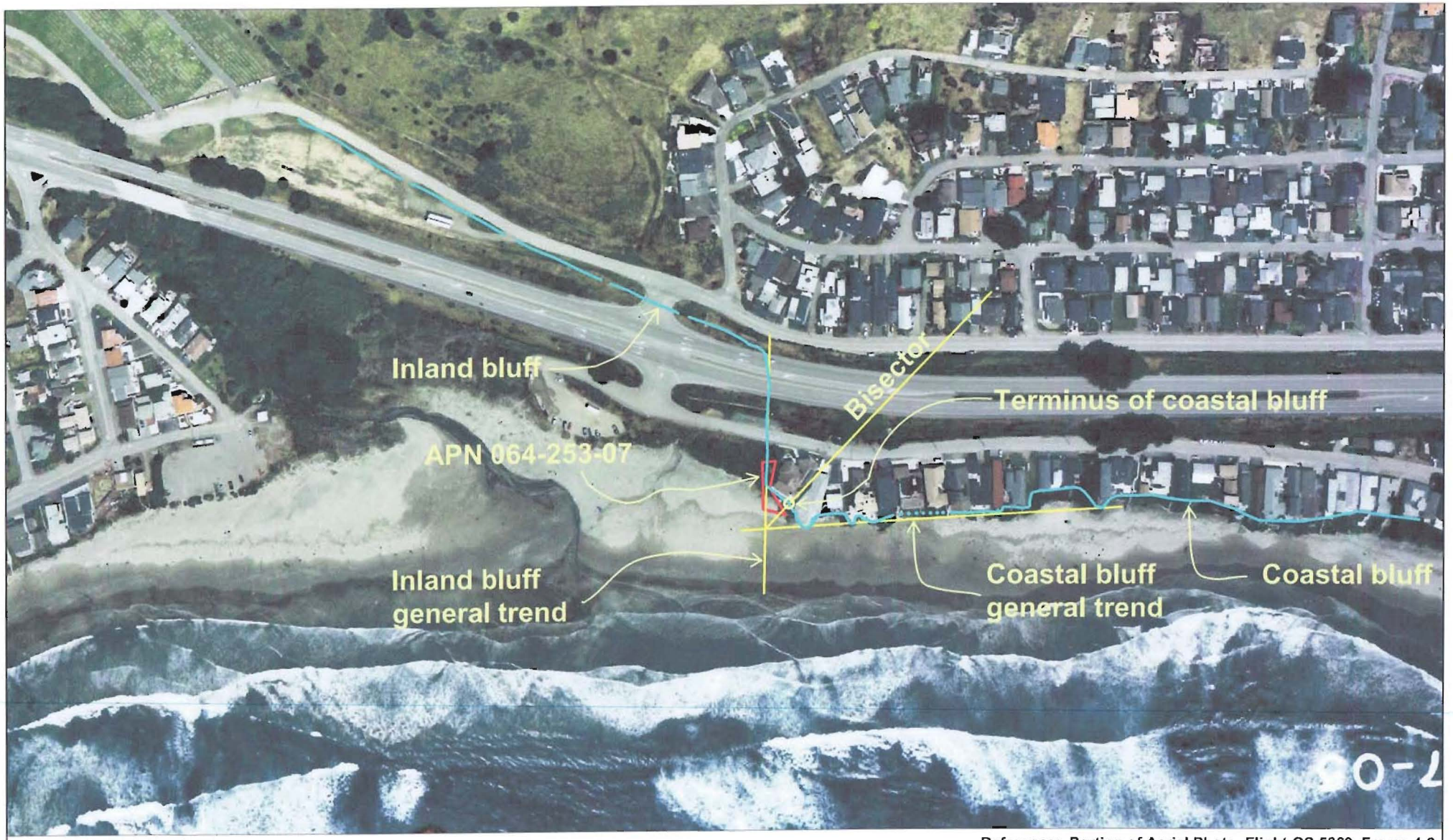
Reference: Portion of Aerial Photo AXH-37
Frame 211, Flown 1937

0 300 ft 600 ft
1"= 300 ft

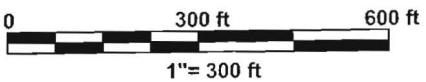
COTTON, SHIRES & ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

1937 Aerial Photo Features
Loperena Property
APN 064-253-07, Studio Drive
CAYUCOS, CALIFORNIA

GEO/ENG BY	SCALE	PROJECT NO.
SS	1 inch = 300 ft	SC0099
APPROVED BY	DATE	FIGURE NO.
MP	May, 2011	5



Reference: Portion of Aerial Photo, Flight GS 5869, Frame 4-2
Flown May 7, 2005



COTTON, SHIRES & ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

Bluff Edge Delineation on 2005 Aerial Photo
Loperena Property
APN 064-253-07, Studio Drive
CAYUCOS, CALIFORNIA

GEO/ENG BY	SCALE	PROJECT NO.
SS	1 inch = 300 ft	SC0099
APPROVED BY	DATE	FIGURE NO.
MP	May, 2011	6

particular case we consider the 300-foot segment of inland bluff to be sufficient for differentiation insofar as it is perpendicular to the shoreline and is thus inland-facing.

In summary, based on our interpretation and application of the CCC guidelines for CCR Title 14, Section 13577, the Loperena property is not located on a coastal bluff.

4.0 HAZARDS REVIEW

In the following sections, we list identified potential geologic/geotechnical hazards and coastal hazards at the project site, along with corresponding degrees of determined potential risk, and recommendations for possible mitigation measures where appropriate.

4.1 Geologic/Geotechnical Hazards

4.1.1 Earthquake Fault Rupture – The project site is not located within an Alquist-Priolo Earthquake Fault Zone as defined by maps prepared by the California Geological Survey. No known active faults trend through the property. Cleath (2006) notes that no topographic anomalies in the area are suggestive of faulting. Based on our review, we concur and therefore consider the potential for surface faulting and ground rupture at the site to be **low**.

4.1.2 Seismic Hazards -- Seismic ground shaking associated with a large earthquake on one of several nearby and regional faults (the Oceanic, Hosgri, Los Osos, and San Luis Range faults) is considered to be a **high** potential hazard for the project area. Peak ground accelerations up to 0.35g could potentially affect structures at the site in the future (see report Section 2.4 Seismic Setting). Mitigation of seismic hazards due to strong ground motion is addressed through proper structural design in accordance with the applicable building codes (presently the 2009 IBC/2010 CBC documents related to Earthquake Loads) at the time of building permit application. Due to the age of the reviewed reports, the seismic ground motion information and code-required design parameters provided by Cleath should be updated as part of an update report that would typically be prepared prior to completion of the structural engineering design for the residence.

Seismically-induced ground failure mechanisms include: landsliding, liquefaction, lurching, differential compaction, lateral spreading, and dry sand settlement. These are discussed as follows:

Earthquake-Induced Landsliding - The central coast region of California has not yet been mapped by the California Geological Survey under the Seismic Hazards Mapping Act program. No landslides have been mapped or found on the property. The only significant slope that would exist at the site upon completion of the project is the fill slope descending from Studio Drive to the property; however, the plans indicate this slope will be filled over and supported by retaining walls; hence we consider the potential for seismically-induced landsliding to be **low**.

Liquefaction - Soil liquefaction is a phenomenon in which a saturated, cohesionless, near-surface soil layer loses strength during cyclic loading (such as typically generated by earthquakes). During the loss of strength, the soil acquires "mobility" sufficient to permit both horizontal and vertical ground movements. Soils that are most susceptible to liquefaction are clean, loose, saturated, uniformly graded, fine-grained sands that are generally located within 50 feet depth beneath the ground surface. Gravels with similar characteristics and non-plastic clays and silts have also been shown to be susceptible to liquefaction.

The applicant's consultants have presented opinions regarding potential site liquefaction. Cleath (2006) opines that the geologic setting and property stratigraphy are such that liquefaction will not occur. GSI Soils, Inc. (2007) notes that due to the presence of relatively shallow bedrock materials and the absence of groundwater, the potential for liquefaction would be in the negligible category. GSI did note, however, that perched water conditions should be anticipated during wet winter months in the upper 5 feet of soils above the dense bedrock materials. GSI has recommended drilled piers extending into the greywacke sandstone bedrock for foundation support for the residence. We are concerned that potentially liquefiable materials (e.g., SP materials with blow count N=7 in boring B-3 at 10′ depth, and other areas of loose soils) cap the bedrock and would be penetrated by the pier foundations if left in place. While some of these potentially liquefiable materials would be removed due to cuts to reach the basement floor subgrade elevation; it is unclear whether GSI recommends removal and recompaction of the remaining loose and potentially liquefiable materials. Hence,

without additional information, we would consider the potential for liquefaction to adversely impact the project to be **moderate** to **high.** As such the consultant needs to clarify their recommendations and either further address the liquefaction potential and its impacts to the proposed development, including foundation elements, or provide recommendations that result in mitigation of the potentially liquefiable materials. The condition can be successfully addressed and mitigated via implementation of typical geotechnical recommendations for site processing, grading, and/or foundation design.

Ground Lurching and Differential Compaction - The potential for lurching and differential compaction (densification) of the existing undocumented fill is considered to be **high** due to the generally loose nature of the soil. This condition can be mitigated by removal, and/or removal and backfilling as structural fill. It is unclear if GSI (2007) is recommending removal of all of the existing fill material (and garbage) underlying the site. Recommendation 1 under Section 5.1 Clearing and Stripping (GSI, 2007) states "All surface and subsurface deleterious materials should be removed from the proposed building area and disposed of off-site. This includes, but is not limited to any buried utility lines, loose fills, septic systems, debris, building materials, and any other surface and subsurface structures within proposed building areas. Voids left from site clearing, should be cleaned and backfilled as recommended for structural fill." The project geotechnical engineer needs to clarify whether this applies to all existing fills within the proposed development area.

Lateral Spreading - The potential for lateral spreading is not addressed in the reviewed reports. Conditions that typically induce lateral spreading include liquefaction of a subsurface layer or layers of soil, and site topography that contains an open topographic face which exposes the soil profile overlying the liquefiable layer(s). Both conditions potentially exist at the site but require further review by the applicant's consultants. The project geotechnical engineer should address the lateral spreading potential in conjunction with their review/updating/clarification requested above for other seismically-induced ground failure mechanisms. For the subject project, we anticipate that lateral spreading potential, if present, can typically be mitigated through geotechnical design recommendations for site processing, grading, and/or foundation design.

Dry Sand Settlement – The potential for seismically-induced dry sand settlement is not addressed in the reviewed reports. The project geotechnical engineer should analyze dry settlement potential of loose sand from seismic ground shaking and discuss risks and mitigations. We anticipate that potential adverse effects from dry sand settlement, if present, can typically be mitigated through geotechnical design recommendations for site processing, grading, and/or foundation design.

4.1.3 **Expansive Soil** -- A single expansion index test was conducted by GSI (2007) on a sandy clay sample from boring B-2 at 6 feet. The reported expansion index was 92, which indicates a high expansion potential. As noted above in Section 3.2, the material in B-2 at this depth is likely weathered mudstone bedrock. The reviewed reports do not specifically address this result or the potential for expansive soils to impact the proposed development. The surficial soils capping the bedrock at the site were reported as sandy clays, clayey sands, and silty sands. Highly expansive soils, if present beneath structures and hardscape or behind retaining walls, could be subjected to volume changes due to irrigation and/or seasonal fluctuations in moisture content of the near-surface soils. Thus, without additional information, we consider there is a **moderate** to **high** potential for expansive soils to impact the site. The project geotechnical engineer should discuss whether expansive soils exist elsewhere at the site, whether these materials may be reused as structural or non-structural fill, and whether additional mitigation measures for expansive soils are necessary. Expansive soil impacts can typically be mitigated through normal geotechnical design recommendations for these conditions. Typical mitigation might range from special foundation/slab design, to removal of expansive soils and replacement with structural fill consisting of non-expansive import soils.

4.1.4 **Landslides** -- Based on our review of published geologic hazard maps (Figure 4) and aerial photographs of the area, there are no landslides mapped on or immediately adjacent to the subject site. A large earthflow landslide terminates approximately 400 feet northeast of the site across Highway 1. This landslide was discussed by Cleath (2006), has been investigated by others (GeoSolutions, Inc., 1999, 2005a, 2005b) and was peer reviewed by CSA for San Luis Obispo County (Cotton, Shires and Associates, Inc., 2007). The landslide and the Loperena property are separated by over 400 feet of very low gradient topography that is overall flatter than 15:1 (horizontal:vertical). Significant portions of that horizontal distance are nearly

level (e.g., the width of Highway 1). Consequently the potential for risk of landslides adversely impacting the site is considered to be **low.**

4.1.5 Slope Instability

Long-term - Construction of the proposed driveway will result in structural fill placement against the existing 2:1 gradient fill slope of Studio Drive, with the fill being supported by retaining walls. Upon completion of the project, no significant slopes will exist that could pose a slope instability hazard to the property. Significant scour of beach sand due to heavy surf may temporarily create a steep bedrock slope oceanward of the existing bedrock outcropping. Provided the proposed residence is constructed on deepened pier foundations as recommended by the project consultants, temporary beach scour should not pose a slope instability hazard to the residence.

Short-term - Based upon our review of the consultant reports, the potential for instability of temporary (construction) slopes is a significant concern. Construction cuts for basement retaining walls may exceed 12 feet depth on the south and east sides of the proposed residence. The applicant's geotechnical engineer (GSI, 2007) indicates that temporary cuts of 1:1 gradient will stand for the short term; however, based on the presence of loose fill materials and depending upon whether those materials will be left in place or removed and recompacted, 1:1 gradient temporary slopes may be too steep. All of the exploration was confined to the north half of the site. Hence, there is also a lack of subsurface geotechnical information along the south property line where temporary stability is critical due to proximity of the existing development on the adjacent property, and at the southeast corner of the proposed basement, where temporary cuts are anticipated to be high. Consequently, there is a **moderate to high** potential for temporary slope instability impacting the project and/or adjacent property. That potential can be mitigated through additional subsurface exploration in the critical areas and supplemental recommendations for temporary construction stability, as appropriate, from the project geotechnical engineer. The project engineering geologist should extend the existing cross-sections A-A′ through C-C′ southeast (or new cross-sections should be prepared) through the adjacent property, depicting the location of existing structures and illustrating the proposed basement excavation in relation to the subsurface soil and bedrock conditions.

4.1.6 Environmental Hazards from Earth Materials – Some areas of California contain ultramafic rocks that also contain naturally occurring asbestos. These include

serpentinite rocks found within the Franciscan Formation in the Cayucos area. The Loperena property has been found to be underlain by greywacke sandstone bedrock with minor interbeds of shale. Cleath (2006) notes that the site is not underlain by earth materials known to contain naturally occurring asbestos. The site is also in an area defined as having a low potential for indoor radon gas levels to exceed the EPA action level of four picocuries per liter of air (Churchill 2008). Consequently, the potential risk from environmental hazards from earth materials, such as naturally occurring asbestos, and radon gas, is considered to be **low**. Mitigation for these conditions is not required.

4.1.7 **Flooding and Drainage** – Since the completion of Whale Rock Dam and Reservoir in April 1961, the potential flood hazard on Old Creek has been substantially reduced. The dam captures water from a 20.6-square mile watershed. Between 1961 and 1998, Whale Rock dam had spilled 8 times (City of San Luis Obispo 1998) but it is apparent that none of these events resulted in flood inundation at the subject property. The site *is* located within the flood inundation zone in the event of failure of Whale Rock Dam (County of San Luis Obispo, Safety Element Dam Inundation Map, 2000); however, this factor is not a restriction to development.

Cleath (2006) notes that the property is not located within a 100-year flood hazard zone according to the San Luis Obispo Local Hazard Mitigation Plan (November 2005, Revision 1). This information appears to remain current. Based upon our review of the latest FEMA FIRM map for Cayucos, the area of the Loperena property proposed for development is located above and outside the AE/VE hazard zone which has a 100-year flood elevation of 10 feet (NGVD29), which is approximately equivalent to elevation 12.92 feet NAVD88. The proposed basement finish floor elevation of 15 feet NAVD88 is approximately 2.08 feet higher than the AE/VE flood elevation.

With regard to local drainage conditions, runoff from the oceanward side of Studio Drive drains down a concrete overside drain and discharges at the toe of the fill slope supporting Studio Drive. This discharge, as well as any runoff from incidental rainfall within the County right-of-way, reaches a natural drainage swale that flows around the northeast corner of the Loperena property and ultimately discharges on the beach. In the proposed development condition, an impermeable (concrete) driveway will cover the area of the County right-of-way, which would apparently result in an increase in surface runoff (less potential for rainfall infiltration due to impermeable surface).

Rainfall from the residence roof is proposed to be collected by a roof gutter system and held in a cistern for gray water use and landscape irrigation. The proposed site drainage improvements will convey both the Studio Drive runoff and the driveway runoff to a drainage exit structure, which outlets into the aforementioned natural drainage swale. The preliminary exit structure design shown on the Plot Plan (Shoreline Engineering 2005) incorporates bollard-style energy dissipators and a gravel/cobble invert, both of which appear aimed at reducing flow velocity and erosion potential. The earth materials in the natural drainage channel consist of highly erodible sands. Erosion in the channel has been accelerated by foot traffic from people accessing Morro Strand State Beach from Studio Drive. The potential increase in surface runoff due to the driveway construction, which increases the potential for erosion in the natural drainage swale, can be mitigated through appropriate civil engineering drainage design. Cleath (2006) also suggests monitoring the condition of the swale while preventing foot traffic, or conveying the aforementioned runoff by a pipe or culvert downslope to the beach.

4.2 Coastal Hazards

The documents that address coastal hazards for the project include reports by Cleath (2006, 2007b), and information submitted by the applicant including Table 2, Summary of Elevations from the FEMA Flood Insurance Study for San Luis Obispo County, California (revised February 4, 2004). Those documents presented the following information with regard to coastal hazards:

- The greywacke sandstone outcrop forms a buttress providing protection from wave action for the landward portion of the site (Cleath 2006).
- Wave runup is expected to reach the sandstone outcrop during spring tides and high tides associated with storm surf conditions (Cleath 2006).
- A site specific study in 1981 estimated a coastal erosion rate of 0.6 in/year for the sandstone materials exposed in the outcrop (Cleath 2006).
- The 100-year and 500-year tsunami runup elevations are 9.5 feet and 24.2 feet respectively based on regional information (County of SLO Safety Element, 1998) (Cleath, 2006; 2007b). We presume these elevations are NGVD29, because the basis for the elevations is a study conducted in 1978.

- Cleath (2007b) indicated that a storm surge of 4.5 meters (~14.5 feet) is the design runup factor that should be used in reference to flooding and inundation standards in the County Code. Cleath (pers. comm., 2011) indicated the basis for this information was Houston and Garcia (1978), and Kilbourne and Mualchin (1980).
- The portion of the 2004 FEMA Flood Insurance Study submitted by the applicant identifies the nearest calculated wave runup elevations, at Cayucos Creek and Little Cayucos Creek, as being 11.4 feet and 20.0 feet for 100-year and 500-year events, respectively.

Based upon our review of this information with assistance from our subconsultant coastal engineer, David W. Skelly, PE of GeoSoils, Inc., we felt that much of the referenced information was regional, outdated, and that there was a strong necessity for preparation of site-specific hazards analyses that address wave run-up, breaking waves, and consideration of future potential shoreline changes due to erosion and sea-level rise over the design life of the project (75 to 100 years). As such it was recommended that a supplemental coastal hazards study be prepared and incorporated into the project EIR. The optional task for this study was authorized by the County, and was performed by our subconsultant, David W. Skelly, PE (GeoSoils, 2011). The report by Skelly is appended to this document (Appendix A), and includes a worst-case analysis of wave run-up conditions incorporating a potential sea level rise of 2 feet over the next 100 years. The report evaluates four different potential oceanographic hazards at the Loperena site: shoreline erosion, flooding hazard due to water level changes in the ocean, breaking wave elevation, and wave runup. These are summarized as follows:

4.2.1 **Erosion Hazard** -- The report cites a 2006 USGS study which concluded that the shoreline in front of the subject property has been relatively stable over the long term. On the basis of the USGS study, aerial photograph review spanning 39 years, the elevation of the proposed development, and the presence of hard rock material between the shoreline and the proposed residence, the report concludes that:

- **there has been very little erosion or retreat of the shoreline over the last four decades;**
- **a 2-foot rise in sea level will likely not result in a significant impact on the erosion rate or the proposed residence; and,**

- **there is no potential significant marine erosion hazard at the site over the next 100 years.**

4.2.2 <u>Oceanographic Flooding Hazard</u> -- The report indicates that the primary hazard due to flooding from ocean waters is super-elevation of the ocean (aka, storm surge). The report notes that the highest recorded water elevation on record in the vicinity of Cayucos (Port San Luis) is 7.57 feet NAVD88 and includes all oceanographic effects on sea level except for long term sea level rise predictions. Incorporating a potential sea level rise of 2 feet in the next 100 years, a future design maximum sea level of 9.6 feet NAVD88 is determined and is considered to be in excess of a 100-year recurrence interval water level. As the site improvements are planned at and above an elevation of 15.0 feet NAVD88, the report concludes that the site is safe from flooding from the ocean over the next 100 years.

4.2.3 <u>Breaking Wave Elevation</u> -- Because the project design incorporates a cantilevered design, wherein the proposed first floor at elevation +26 feet NGVD88 will extend a significant distance oceanward beyond the basement floor, the report evaluated the potential maximum breaking wave crest elevation. The breaking wave elevation analysis calculated that the maximum wave crest elevation at the proposed structure for the subject site is about +14 feet NAVD88, which is well below the proposed cantilevered first floor elevation of +26 feet NAVD88; hence, the cantilevered portion of the structure appears to be safe from breaking wave forces.

4.2.4 <u>Wave Runup Hazard</u> – A wave runup analysis was performed under extreme (worst-case) design oceanographic conditions including super-elevation of the ocean, sea level rise of two feet over the next 100 years, and scour of the beach in front of the rock outcropping down to elevation 0.6 feet NAVD88, utilizing an extreme wave height of 7 feet. The report calculates a maximum wave runup in these extreme conditions to elevation +22.2 feet NAVD88 and concludes that wave runup may reach the basement of the proposed residence at +15.0 feet NAVD88 over the next 100 years in this scenario. However, the runup is characterized as a pulse of water reaching the basement wall rather than a continuous or sustained flow over time. The runup analysis indicates that the velocity of the wave runup bore will not be sufficient to cause damage to the structure, assuming the basement wall is constructed of steel-reinforced concrete; however, the structure will be subject to spray and splash from wave runup

striking the rock outcropping. The report concludes that wave runup will not adversely impact the proposed residence over the next 100 years due to the structure elevation, under the assumed steel-reinforced construction of the basement. The report also notes, based upon review of historical data and tsunami forecast modeling by the USC Tsunami Research Center, that a 6.5 feet high tsunami wave for the area of the subject site would be on the order of a 500-year recurrence interval event; hence, the report considers that the wave runup analysis that was performed using a design wave height of 7 feet also represents a suitable site-specific tsunami runup at the site.

4.3 Hazards Summary/Conclusions

Based upon our review of the proposed project, we find that geologic/geotechnical hazards that have a moderate or high potential for impacting the project include seismic shaking and other seismically-induced ground failure mechanisms; expansive soils; instability of temporary (construction) slopes; and local offsite erosion from the site drainage design. All of these items can be addressed and mitigated via implementation of recommendations provided by the project geotechnical consultants and/or design civil engineer, and compliance with current building codes and local ordinances governing construction.

The engineering geologic and geotechnical engineering reports for the project are 4 to 5 years old, and thus require updating. In addition, we have identified a number of deficiencies in the reviewed reports where additional data, information, analysis, and recommendations should be provided so that this information can be incorporated into the project EIR.

Based upon the supplemental coastal hazards study performed, our coastal engineering subconsultant concludes that there are no recommendations necessary to mitigate potential coastal hazards, and the proposed residence will neither create nor contribute to erosion, geologic instability, or destruction of the site or adjacent area as a result of coastal hazards/processes.

5.0 RECOMMENDATIONS

Based upon our peer review of the project engineering geologic and geotechnical engineering documents, project plans, site observations, review of aerial photographs, and hazards review, we make the following recommendations for your consideration.

5.1 Update Reports

The project geotechnical professionals (Cleath & Associates for engineering geology and GSI Soils, Inc. for geotechnical engineering) should be required to prepare update reports that include a detailed review of the project plans. The reports should address the peer review comments presented in Section 1.6 as well as deficiencies in information that are described in Section 4.0, Hazards Review. Based on our review, additional subsurface exploration and laboratory testing of soil and bedrock materials is warranted. In our opinion, it is important that preparation of the update reports be completed now, so that this information can be incorporated into the project EIR. In this manner, the various potential hazards identified can be reduced to insignificant levels through implementation of typical practice geotechnical design recommendations, incorporation of those recommendations into the project plans, code compliance, and proper construction. Preparation of the update reports is also critical for proper input to the preparation of detailed construction plans, including foundation and retaining wall plans/details and accompanying structural engineering calculations, as well as excavation/grading and site drainage plans. Update reports should be peer reviewed by the County of San Luis Obispo or its contracted subconsultant.

5.2 Site Drainage Plan

The project civil engineer (Shoreline Engineering) should prepare an updated site drainage plan, and/or provide a confirming statement (supported by hydrology calculations if necessary) that the combined runoff from Studio Drive and the proposed driveway will not significantly increase erosion in the drainage swale that leads down to the beach. Alternative recommendations should be provided if necessary to mitigate any increased erosion potential identified. It should be anticipated that alternatives such as extending the stormwater discharge through non-erosive devices down to the beach, will require review and approval by those having authority over Morro Strand

State Beach. Those same authorities should consider measures to discourage foot traffic in the swale by pedestrians accessing the beach from Studio Drive.

5.3 DEIR Review

CSA recommends that we be afforded the opportunity to review and comment upon the Draft Environmental Impact report for the project.

6.0 LIMITATIONS

Our services consist of professional opinions and recommendations made in accordance with generally accepted engineering geology and geotechnical engineering principles and practices. No warranty, expressed or implied, or merchantability of fitness, is made or intended in connection with our work, by the proposal for consulting or other services, or by the furnishing of oral or written reports or findings.

Any recommendations presented in this report are contingent upon our firm being retained to review and comment on the Draft Environmental Impact Report.

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7.1 Aerial Photographs Reviewed

California Coastal Records Projects-Aerial Photographs of the California Coastline; url: http://www.californiacoastline.org, images reviewed:

Image No.	Date	Type
7226002	1972	color oblique
7938052	1979	" "
198630254	1/19/1987	color vertical
198630255	1/19/1987	" "
8901175	1/1989	color oblique
199300097007	6/25/1993	color vertical
199300097008	6/25/1993	" "
2042	9/2002	color oblique
200403467	10/2004	" "
200509737	10/2005	" "
200509738	10/2005	" "
20087247	9/2008	" "
20106688	9/2010	" "

Golden State Aerial Surveys, San Luis Obispo, CA
Color Vertical Aerial Photographs:

Flight I.D.	Date	Frame Numbers
GS CY88	12-30-1988	1, 2
GS 5869	5-7-2005	4-1, 4-2

University of California, Santa Barbara Map & Imagery Library, Santa Barbara CA, Black and White Vertical Aerial Photographs:

Flight I.D.	Date	Frame Numbers
AXH 1937	1937	209, 210, 211
AXH 1949	1949	5F-113, 114, 84, 85
AXH 1956	1956	57, 58
HA-GI	1959	112, 113, 114
HA-VG	1963	17, 18, 25, 26

APPENDIX A

Discussion of Coastal Hazards and Wave Runup, Northwest and Immediately Adjacent to 2612 Studio Drive (APN 064-253-07), Cayucos, San Luis Obispo County, California, by GeoSoils, Inc., March 14, 2011

GeoSoils Inc.

March 14, 2011

Mr. Michael Phipps
Cotton Shires and Associates, Inc.
550 St. Charles Drive, Suite 108
Thousand Oaks, CA 91360-3995

Subject: Discussion of Coastal Hazards and Wave Runup, Northwest and Immediately Adjacent to 2612 Studio Drive (APN 064-253=07), Cayucos, San Luis Obispo County, California

Dear Mr. Phipps:

At your request and authorization, GeoSoils Inc is pleased to present the following report describing the coastal hazards and wave runup associated with the proposed residential structure development at subject site. The analysis is based upon existing published reports concerning the regional coastal processes, site specific geology and geotechnical reports, site elevations, and knowledge of coastal conditions. This report constitutes an investigation of the wave and water level conditions expected at the site in consequence of extreme storm and wave action. The purpose of this report is to provide the necessary coastal engineering information for inclusion in the project EIR and to obtain a permit from the County of San Luis Obispo and the California Coastal Commission for the construction of a new residence. It provides conclusions and recommendations for the susceptibility of the proposed development to wave attack shoreline erosion and flooding.

SCOPE OF SERVICES

1. Review of available regional and site-specific oceanographic and geotechnical reports.

2. Research of historical aerial photographs of the site, using historical aerial photographs to determine historical shoreline changes.

3. Engineering analysis and preparation of this report that includes a review of the available oceanographic information, a discussion of past and future erosion rates, calculation of wave runup, discussion of site flooding from the ocean, and conclusions and recommendations.

SITE DESCRIPTION

The subject site is an irregularly shaped parcel at the northern end of Studio Drive. Site elevations vary from about +10 feet NAVD88 at the beach area to about elevation + 30 feet NAVD88 at Studio Drive. The majority of the parcel is at or above elevation +20 feet NAVD88. The site is fronted by a bedrock outcropping (Graywacke Sandstone) from about elevation +17 feet NAVD88 to the beach at about elevation +10 feet NAVD88. This bedrock serves as a form of natural shore protection. Photograph 1 is an aerial photograph taken in September 25, 2010 downloaded with permission from the California Coastal Records Project web site (http://www.californiacoastline.org/).

Photograph 1. Subject site and adjacent development and shoreline September 25, 2010.

WAVES & WATER LEVELS

Waves of all periods approach the Cayucos shoreline; however, almost all of the energy is contained in the medium and long period waves(approximately 5 to +20 seconds). These waves can approach from the north, the west, and even the south. As waves travel into shallower and shallower water, the wave crest is bent and becomes nearly parallel to shore, and the wave heights are modified depending on whether waves are being focused or de-focused at a particular location along the shoreline. This process is called refraction and it is dependent upon the bathymetry, and the wave height, period, and direction. The California Department of Boating and Waterways in partnership with the US Army Corps of Engineers maintain wave recording buoys throughout the central California coast in the Coast Data Information Program (CDIP). The closest long term continuous wave recording buoys to the site are CDIP Buoy 076 located near Diablo Canyon and CDIP Buoy 157 at Point Sur. The record of extreme waves for this region from these buoys covers as far back as 1978 with extreme waves in excess of 35 feet and with periods in excess of 20 seconds recorded during the 1982-83 El Niño winter.

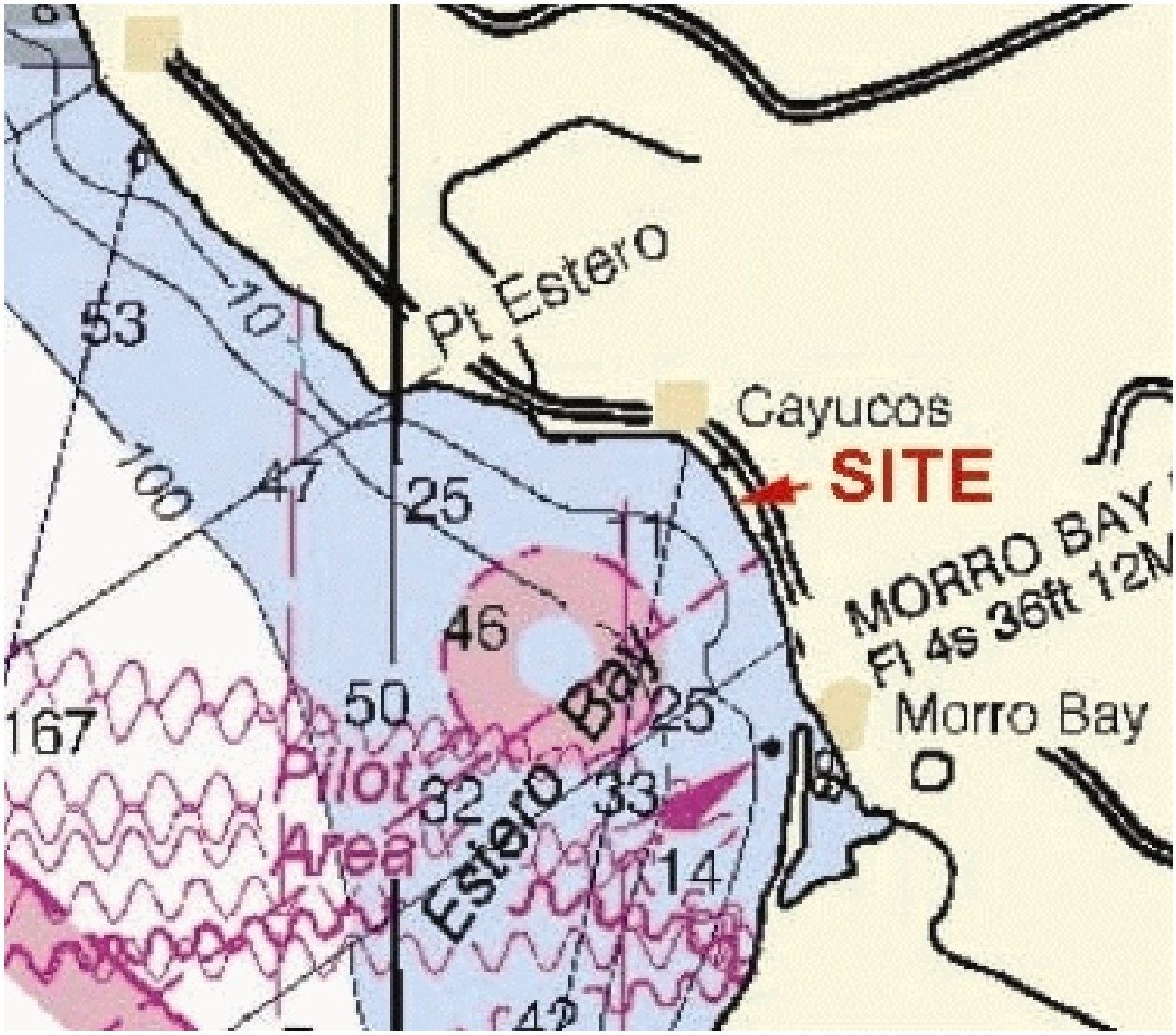
The National Oceanographic and Atmospheric National Ocean Survey (NOAA, 2011) operational tidal data station closest to Cayucos is located at Port San Luis (Station 9412110). The tidal datum elevations for the 1983-2001 epoch are as follows:

Highest Water Jan 18, 1973	7.57 feet
Mean Higher High Water	5.25 feet
Mean High Water	4.54 feet
NGVD29	2.93 feet
Mean Low Water	0.96 feet
NAVD88	0.0 feet
Mean Lower Low Water	-0.08 feet

OCEANOGRAPHIC DESIGN PARAMETERS

There are several factors that are important to the analysis of the vulnerability of a structure (residence) along the shoreline. Some of the factors are based upon the existing topography/bathymetry and elevation of the proposed structure at the site. Based upon NOAA Bathymetric Chart 18022, the site lies within Estero Bay with relatively slight slopes to deep water, see Figure 1. The offshore, from 0.0 NAVD88 to ~-60 feet NAVD88 is relatively flat at 1/100 (V/H). Site elevations relative to NAVD88 were provided by Volbrecht Surveys. The beach fronting the site is relatively flat and the rock outcropping fronting the site rises from +10 feet NAVD88 to about +17 feet NAVD88 in about 15 feet

horizontal distance. Other factors are based upon extreme oceanographic conditions or the coincidence of several extreme conditions. In order to determine design wave characteristics for the runup and breaking wave elevation analysis, it is necessary to determine the design water level. The design water level will need to account for the



future rise in sea level over the life of the structure.

Figure 1. NOAA Bathymetirc Chart 18022 updated in September 2008. The depths are in fathoms.

Sea Level Rise

The current EPA sea level rise prediction is available on the EPA website and provided herein as Figure 2. The EPA approximate range for sea level elevation in 2100 is 210 mm (8.3 in) to 500 mm (19.7 in) above present sea level. More recently published scientific papers suggest that this magnitude of sea level rise is at the low end of the possible range. For this analysis, future sea level rise for the site was determined from the Cayan, et al., 2008 scientific paper entitled "Climate Change Projections of Sea Level Extremes Along the California Coast." This paper is specific to the California coast and provides a range in future sea level rise from 11 cm (4.3 in) to 74 cm (29 in) over then next 100 years (see Figure 3).

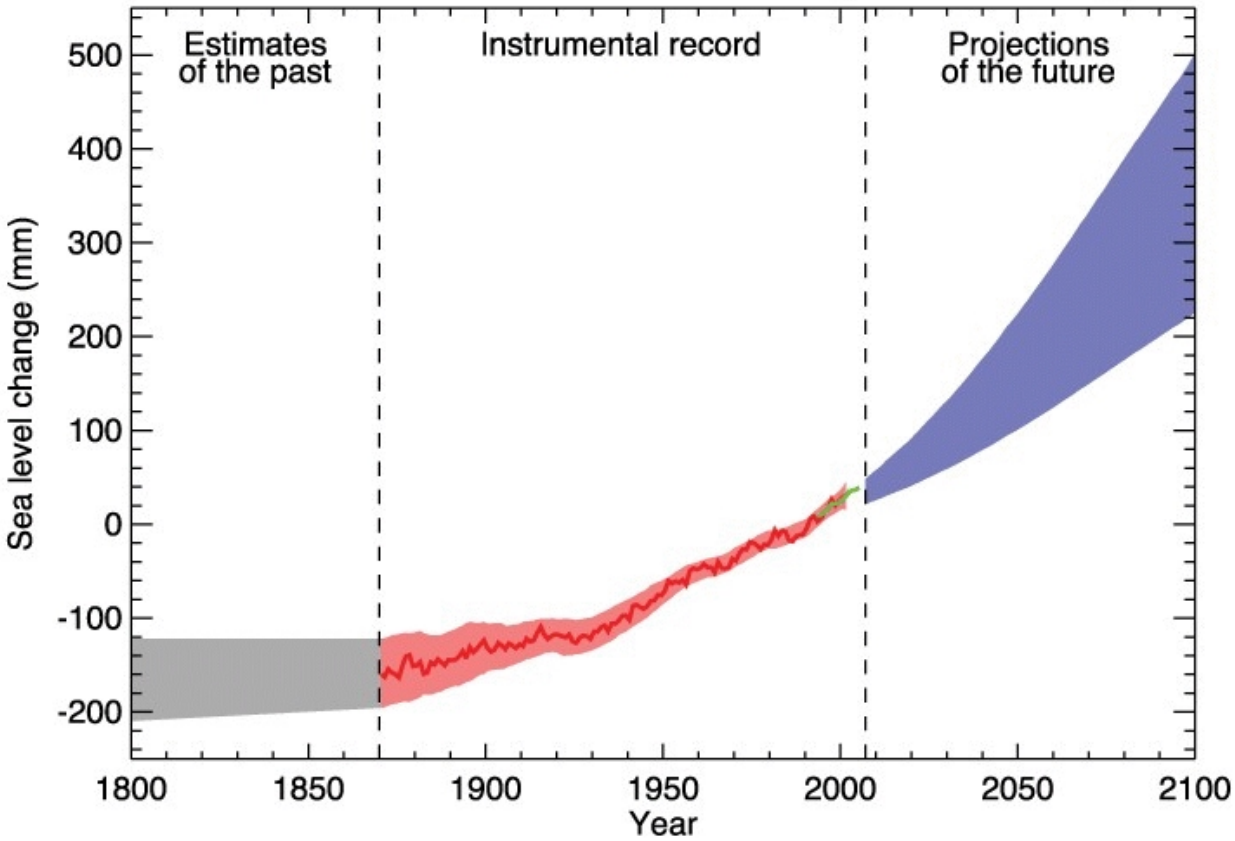


Figure 2. Illustration of the global mean sea level (deviation from the 1980-1999 mean) as observed since 1870 and projected for the future. The future projections have been supplied by the Intergovernmental Panel on Climate Change (IPCC).

Projected sea level rise from climate model estimates for three GHG emissions scenarios, A1fi (high emissions), A2 (medium-high emissions) and B1 (low emissions)

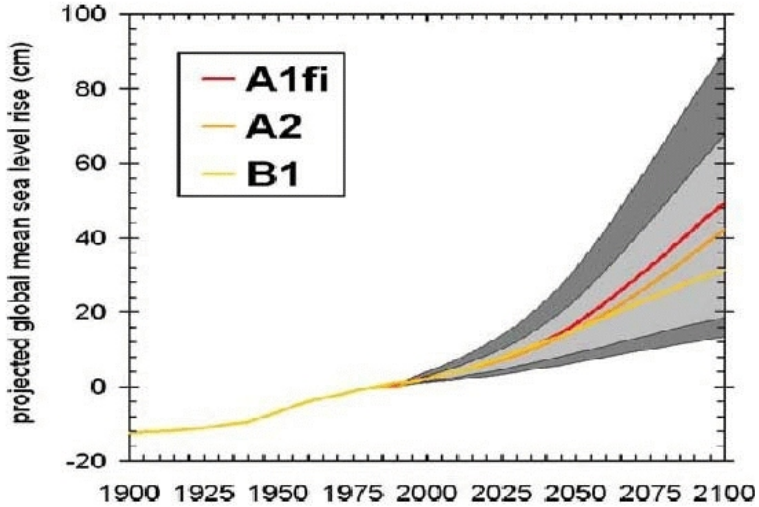


Figure 3. Sea Level rise predictions from Cayan, et. al.

The proposed residential structure has an expected life of 75 to 100 years. Using the EPA estimate, this is a 24 cm to 56 cm range in 100 years. Using Cayan et al., the sea level rise estimate is 30 cm to 80 cm in the next 100 years. To be reasonably conservative and for analysis purposes we will use 65 cm, which is about 2 feet. The highest recorded water elevation on record in the vicinity of Cayucos (Port San Luis) is 7.57 feet NAVD88. This actual high water record covers the 1982-83 severe El Nino. This elevation includes all oceanographic effects (short-term) on sea level except the long term sea level rise prediction. If 2 feet is added to this +7.6 feet NAVD88 elevation a future design maximum sea level of 9.6 feet NAVD88 is determined.

Determination of the maximum scour depth at the toe of the outcropping enables the coastal engineer to determine the actual water depth at the toe of the outcropping and breaking wave elevation under the design water level conditions. The design scour elevation is estimated based upon the erodibility of the materials at the shoreline. A conservative estimate of the scour elevation at the toe of the rock outcropping is about 0.6 feet NAVD88. This is reasonable based upon the visual presence of bedrock at the back shore area. Using the maximum still water elevation and the maximum scour of 0.6 feet NAVD88 yields a total water depth of about 9.0 feet at the eroded beach toe (the rock outcropping). This represents the worst possible conditions wave runup conditions reaching the site over the next 100 years and will be used in the design analysis.

As discussed above, waves from distant storms have pounded the coastline of Cayucos several times within the last few centuries. However, these extreme waves break further offshore and lose a significant portion of their energy before they reach the shoreline. Once a wave reaches a water depth that is about 1.28 times the wave height, the wave breaks and runs up onto the shore. The design wave height is the maximum unbroken wave at the toe of the rock outcropping. The total water depth is 9.0 feet which would yield a design wave height of about 7 feet.

WAVE RUNUP AND OVERTOPPING ANALYSIS

As waves approach the shoreline and the site, they break and water rushes up the rock outcropping, and towards the proposed development. Wave runup is defined as the vertical height above the still water level to which a wave will rise on a structure (the rock outcropping) of infinite height. Overtopping is the flow rate of water over the crest of the outcropping (about elevation +17 feet NAVD88) as a result of wave runup. Wave runup and overtopping for an extreme tsunami event is calculated using the US Army Corps of Engineers Automated Coastal Engineering System (ACES). ACES is an interactive computer based design and analysis system in the field of coastal engineering. The methods to calculate runup and overtopping implemented within this ACES application are discussed in greater detail in Chapter 7 of the Shore Protection Manual (1984) and the Coastal Engineering Manual (2004). The overtopping estimates calculated herein are corrected for the effect of onshore winds. Figure 4 is a diagram showing the analysis terms.

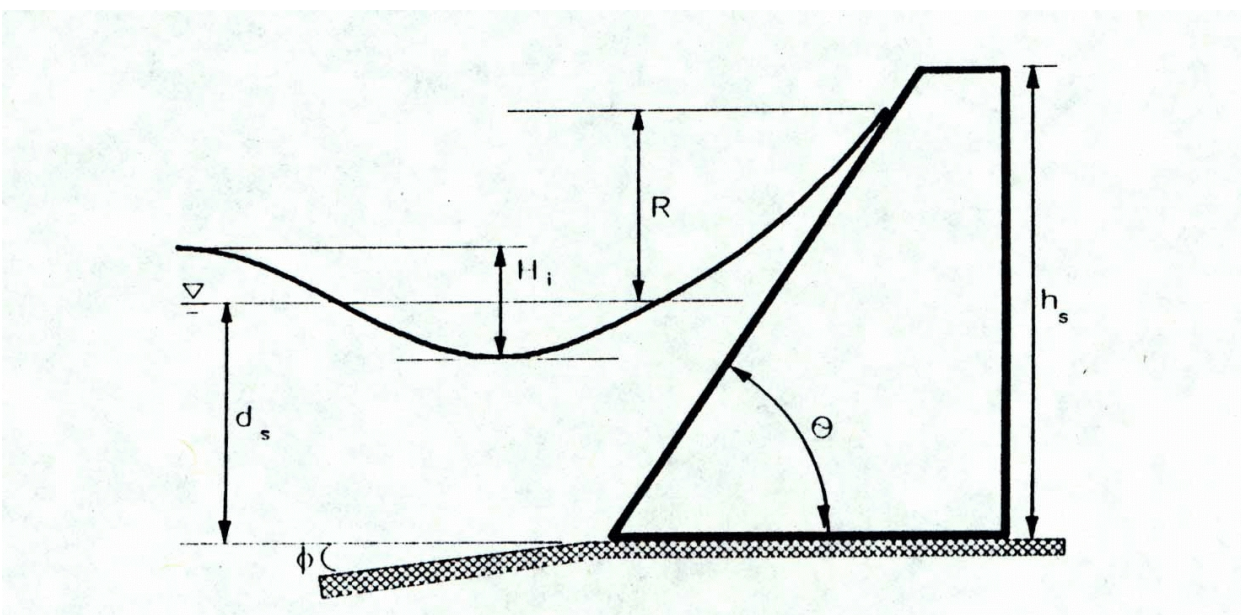


Figure 4. Wave runup terms from ACES manual.

The wave, wind, and water level data used as input to the wave runup and overtopping application will be the extreme wave height of 7 feet, a period of 18 seconds, with the water level at highest recorded water level, corrected for future sea level rise. The nearshore slope is 1/100 from Figure 1 and the outcropping slope is ~1/2 (V/H) from the survey. **Table I** is the ACES output for these design conditions.

TABLE I

AUTOMATED COASTAL ENGINEERING SYSTEM ... Version 1.02 3/14/2011 9:37
Project: 2612 STUDIO DRIVE CAYUCOS WAVE RUNUP

WAVE RUNUP AND OVERTOPPING ON IMPERMEABLE STRUCTURES

Item		Unit	Value	
Wave Height at Toe	Hi:	ft	7.000	Rough Slope
Wave Period	T:	sec	18.000	Runup and
COTAN of Nearshore Slope			100.000	Overtopping
Water Depth at Toe	ds:	ft	9.000	
COTAN of Structure Slope		'	2.000	
Structure Height Above Toe	hs:	ft	17.000	
Rough Slope Coefficient	a:		0.956	
Rough Slope Coefficient	b:		0.398	
Deepwater Wave Height	H0:	ft	4.218	
Relative Height	(ds/H0):		2.134	
Wave Steepness	(H0/gT^2):		0.405E-03	
Wave Runup	R:	ft	12.676	
Onshore Wind Velocity	U:	ft/sec	67.512	
Overtopping Coefficient	Alpha:		0.500E-01	
Overtopping Coefficient	Qstar0:		0.130	
Overtopping Rate	Q:	ft^3/s-ft	0.975	

The rock outcropping at its average elevation of +17 feet NAVD88 can be overtopped by the design wave at a rate of about 1 ft^3/sec-ft. This amount of overtopping is about 1 foot of water per linear foot of outcropping with each wave. The calculated maximum wave runup is the still water elevation of +9.6 feet NAVD88 plus 12.6 feet which is elevation +22.2. It should be noted that above elevation +17 NAVD88 (top of the outcropping) the wave runup/overtopping has very reduced velocity or little or no erosion potential.

These results can also be used to determine potential tsunami impacts at the site, in light of future sea level rise over the life of the structure. There are many experts that study and publish up to date information on tsunamis including Dr. James Lander at NOAA, James Huston, and Dr Jose Borrero, and Dr. Costa Synolakis at the USC Tsunami Research Center (http://www.usc.edu/dept/tsunamis/2005/index.php). Based upon our review of the historical data and tsunami forecast modeling by the USC Tsunami Research Center, a 6.5 feet high tsunami for this area would be on the order of a 500-year recurrence interval event. So the wave runup from the design wave of 7 feet in height is

a suitable site-specific tsunami runup at the site. It should be noted that the recent March 2011 tsunami had no measurable impact on the site.

BREAKING WAVE ELEVATION

Because a potion of the proposed development will actually cantilever out over the rock outcropping the maximum breaking wave crest needs to be determined. The maximum breaking wave crest elevation is important for establishing the maximum vertical extent of wave forces on the improvements. Typical coastal design practice would require that the lowest horizontal structural member be at, or above, the maximum breaking wave crest elevation or be designed to withstand the force of the breaking wave. The maximum breaking wave crest elevation is calculated using the US Army Corps of Engineers Shore Protection Manual (1984) and Coastal Engineering Manual (2004). Figure 5 shows the variables for calculating the maximum breaker height. The variables for the analysis are as follows:

m = the nearshore slope = 1:100 or 0.01.

H_b = the height of the breaker.

d_s = depth of water at the structure = 9 feet.

d_b = depth of water at the break point.

H'_b = the elevation of the breaker

The actual equations for determining the breaker, H_b, and the elevation of the breaker, H'_b, are complex. However, the Shore Protection Manual contains graphs that determine solutions based upon the identified maximum variables. For the purpose of this analysis, a graphical solution will be used. The graph is found on page 7-10 of the Shore Protection Manual, and is reproduced here as Figure 6 for the convenience of the reviewer.

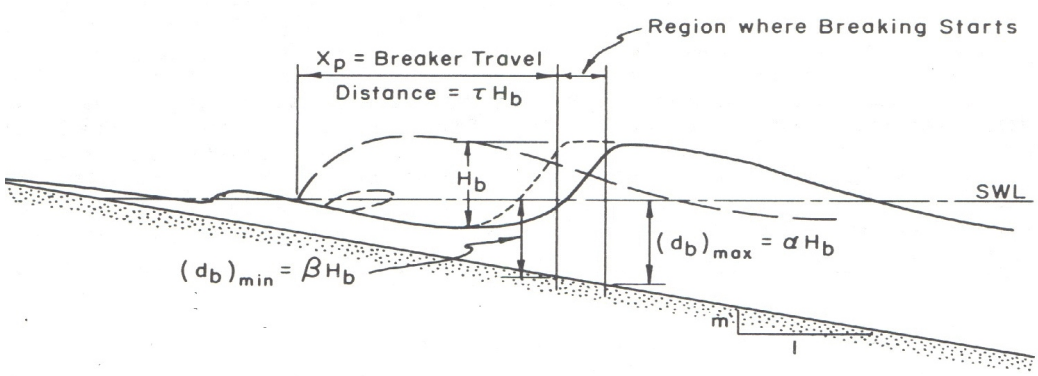


Figure 5. From US Army Corps Engineers Shore Protection Manual Chapter 7, page 7-5, Figure 7-1.

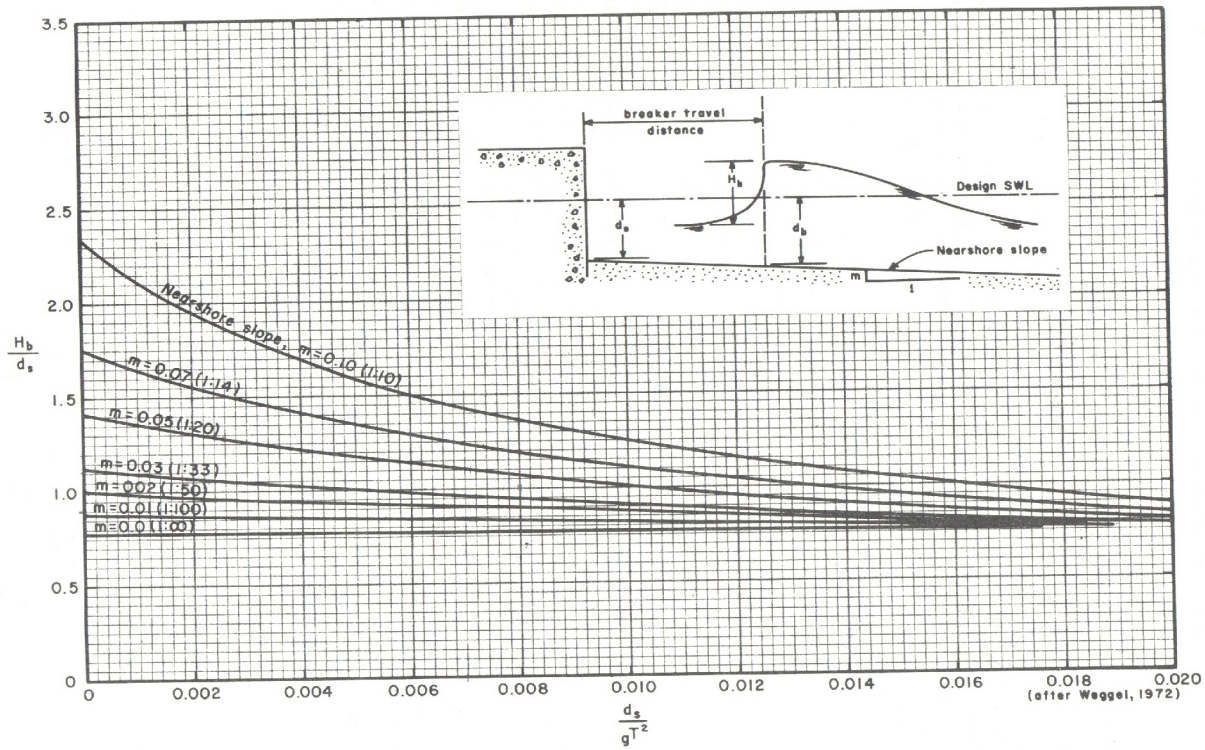


Figure 7-4. Dimensionless design breaker height versus relative depth at structure.

Figure 6. Graphical solution for breaker height from US Army Corps of Engineers Shore Protection Manual.

The maximum breaking wave that can occur at the structure happens when $d_s/(gT^2) = 0.0$. So for m=0.01, H_b/d_s = about 0.7 then H_b = 6.3 feet. A review of Figure 5 reveals that the breaker height is not of equal distribution about the still water line. More of the height is above the still water line than below the still water line. A conservative estimate of the distribution would be that about 70% of the height is above the still water line. So, H'_b = +9.6 feet NAVD88 plus 0.7(H_b) = about + 14 feet NAVD88. <u>The calculated maximum wave crest elevation at the proposed structure for the subject site is about +14 feet NAVD88</u>. This means that the lowest horizontal structural member for the cantilevered portion fo the proposed development should be above +14 feet NAVD88 or designed to resist wave forces.

HAZARD ANALYSIS

There are three different potential oceanographic hazards identified at this site; shoreline erosion, flooding, and waves. For ease of review each of these hazards will be analyzed and discussed separately followed by a summary of the analysis including conclusions and recommendations if necessary.

Erosion Hazard

In an effort to determine typical changes in the shoreline position aerial photographs from the early 1970s to 2010 were reviewed. Due to the hard rock nature of the shoreline material there has been very little erosion or retreat of the shoreline over the last 4 decades. The United States Geological Service in 2006 (USGS 2006) prepared the National Assessment of Shoreline Change Part 3: Historical Shoreline Change and Associated Coastal Land Loss Along Sandy Shorelines of the California Coast, which concluded that the shoreline in front of this site was relatively stable over the long term. Based upon the hard rock material at the shoreline, and the elevation of the proposed development, a 2 foot rise in sea level will likely not result a significant impact on the erosion rate or the proposed residence. There is no potential significant marine erosion hazard at the site over the next 100 years.

Flooding Hazard

The flooding hazard discussed in this section is due to water level changes in the ocean. Flooding due to waters other than from the ocean is mitigated through the site drainage plan designed by the project civil engineer. The primary hazard due to flooding from ocean waters would be due to a super-elevation of the ocean. The National Oceanographic and Atmospheric National Ocean Survey (NOAA, 2011) operational tidal data station closest to Cayucos is located at Port San Luis (Station 9412110). Allowing for a 2 feet rise in sea level over the next 100 years, the mean higher high water will be at about +7.25 feet NAVD88. The highest recorded water elevation on record in the vicinity of Cayucos (Port San Luis) is 7.57 feet NAVD88. This actual high water record covers the 1982-83 severe El Nino winter. This elevation includes all oceanographic effects (short term) on sea level except the long term sea level rise prediction. If 2 feet is added to this +7.6 feet NAVD88 elevation, a future design maximum sea level of 9.6 feet NAVD88 is determined. This would be considered in excess of a 100 year recurrence interval water level. The site improvements are at or above +15 feet NAVD88, which is well above any potential ocean flood elevation. The site is safe from flooding from the ocean over the next 100 years. Potential flooding associated with wave runup is consider in the next section.

Wave Runup

Wave runup may reach the basement of the proposed residence at about elevation + 15 feet NAVD88 over the next 100 years under the infrequent, extreme design oceanographic conditions. Wave runup will actually be a pulse of water reaching the basement wall and not a continuous or sustained flow over time. The basement will be constructed of steel reinforced concrete material. The velocity of the runup bore when it reaches the building will be less than 5.5 ft/sec which is not sufficient to cause any damage to the structure. In addition, the basement and possibly the first floor may be subject to spray and splash from wave runup striking the rock outcropping. Wave runup will not adversely impact the proposed residence over the next 100 years due to the structure elevation and construction. An extreme tsunami may reach as high the basement, but for the reasons stated above, a tsunami will not adversely impact the residence.

CONCLUSIONS

The potential coastal hazards associated with the proposed residential development at Northwest and Immediately Adjacent to 2612 Studio Drive, Cayucos, include shoreline erosion, wave runup, and coastal flooding. Based upon review of historical oblique photographs, the shoreline fronting the site is stable over the long term. During the coincidence of high tides and high waves, the residence may be subject to wave runup. However, based upon our analysis herein, the residence is reasonably safe from coastal hazards. There are no recommendations necessary to mitigate potential coastal hazards. New shore protection will not be required to protect the proposed residence over the next 100 years. The proposed residence will neither create nor contribute to erosion, geologic instability, or destruction of the site or adjacent area.

LIMITATIONS

Coastal engineering is characterized by uncertainty. Professional judgements presented herein are based partly on our evaluation of the technical information gathered, partly on our understanding of the proposed construction, and partly on our general experience. Our engineering work and judgements have been prepared in accordance with current accepted standards of engineering practice; we do not guarantee the performance of the project in any respect. This warranty is in lieu of all other warranties express or implied.

GeoSoils Inc.

Respectfully Submitted,

GeoSoils Inc.
David W. Skelly MS, PE

REFERENCES

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____, January 7, 2007; Addendum to Report of Geologic Conditions at the Loperena Property, Studio Drive (attached to a cover letter from the applicant dated 2/9/07)

____, March 30, 2007; Summary of Conclusions and Recommendations of Cleath & Associates' Study of Geologic Conditions at the Loperena Property, Studio Drive, Cayucos, California

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COTTON, SHIRES AND ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

August 21, 2012
SC0099

Ms. Shawna Scott
SWCA ENVIRONMENTAL CONSULTANTS
1422 Monterey Street, Suite C200
San Luis Obispo, California 93401

SUBJECT: **Supplemental Geotechnical Peer Review for Environmental Impact Report Preparation**
RE: Loperena Minor Use Permit/Coastal Development Permit
Studio Drive, Cayucos, San Luis Obispo County, CA

Dear Ms. Scott:

In accordance with your request, Cotton, Shires and Associates, Inc. (CSA) has performed a supplemental geotechnical peer review of materials submitted for the referenced project. We received and reviewed the following:

- Updated Geotechnical Investigation (GSI Soils, Inc., Dec 27, 2011)
- Updated Engineering Geology Report (Cleath-Harris Geologists Inc., June 25, 2012)
- Site Plan, Sections, Residence Profile and Shoring Detail (Shoreline Engineering, January 2012)
- Floor Plans and Elevations (C.P. Parker Architect, Jan 4, 2012)

The documents referenced above were peer-reviewed from engineering geologic and geotechnical engineering perspectives, with reference to our previous peer review comments and technical report dated May 31, 2011. The purpose of our supplemental review was to determine whether the previous geotechnical peer review comments have been satisfactorily addressed and the additional project information and recommendations from the consultants can be incorporated into the project EIR.

DISCUSSION

Based upon our review of the referenced materials, additional exploration was conducted on the site by the project consultants including two additional borings by GSI Soils, Inc. and four additional borings by Cleath-Harris Geologists, Inc. (CHG). The

consultants also presented new or updated analyses, maps and cross-sections (including updated sections A-A′ and B-B′ by GSI, updated section C-C′ and new section D-D′ by CHG) and updated geotechnical recommendations. It appears that the present concept consists of constructing the residence on a structural mat foundation supported on deepened footings extending to elevations 8 to 11 feet above mean sea level (NAVD88), and constructing a large 6-foot thick concrete deadman footing under the eastern portion of the building footprint. Recommendations are also provided for drilled pier foundations. A soldier beam shoring system with steel traffic plate lagging is indicated on the plans (by Shoreline Engineering), along the southern property line and a portion of the northern property line.

Based on our review of the referenced documents, we recommend that the following additional comments be addressed by the project consultants prior to incorporation of the information and recommendations from the consultants into the project EIR:

Engineering Geology Comments – Review of the Cleath Harris Geologists Inc. report dated June 25, 2012 generated the following comment:

1. Cross-Section B-B′ previously prepared by Cleath (2006) should be updated to reflect the new subsurface data from GSI Soils Inc. (GSI Boring B-5 on the section line) and to have consistency at the intersection of Cleath Cross-Section B-B′ with their new longitudinal Cross-Section D-D′. This seems particularly important since Shoreline Engineering's Shoring Details at SL-1 and SL-2 (Note 4 on Sheets 4 and 5, prepared January 2012) are referencing older information (e.g. "Greywacke sandstone level per Cleath & Harris 5/2/2006") that has since changed as a result of recent subsurface exploration.

Geotechnical Engineering Comments – Review of the GSI Soils Inc. report dated December 27, 2011 generated the following comments:

1. On page 3 of the report it is stated that: "*A sample of the hard sandstone material was tested for compressive strength. The results of this test indicate that the materials have a compressive strength of 1240 psi.*" This value is also reported on the boring log for GSI boring B-5 at 9 feet, for a 2-inch tall retrieved Mod. California Sample; however, no other information regarding the type of test performed was provided in the Laboratory Testing section (Appendix B). Additional information is requested for this test (test sheet, type of test, sample size, length to width ratio, etc.). ASTM D2166 indicates that the height to diameter ratio shall be between 2 and 2.5, and that the minimum diameter shall be 1.3 inches for an Unconfined Compressive Strength test.

2. In Section 4.2 Seismic Settlement of the report, reference is made to the grading recommendations provided in Section 5.2 of the report, noting that said grading

"*… would result in 3 feet of compacted decomposed granite over at least 1 foot of compacted native sands*." However, in Section 5.2 of the report, there is no recommendation to compact 1 foot of the native sands. This should be clarified by the consultant.

3. In Section 5.0 the consultant concludes that "*Due to the presence of relatively shallow bedrock and the absence of groundwater the potential for liquefaction would, in our opinion, be in the low to negligible category.*" We do not agree with this statement because the consultant has determined that under certain seismic and groundwater conditions, there is a potential hazard for liquefaction to occur beneath a portion of the site, and the consultant has provided recommendations to mitigate some of the effects (total and differential settlements) from this hazard so as to not adversely affect the proposed foundation system.

4. The greywacke sandstone bedrock at the site is described by Cleath (2006) as fractured and was observed by CSA to be closely fractured in outcrop, with 2.5 to 8 inch fracture spacing due to the presence of numerous intersecting joint sets. The consultant cites the large uplift loads at the site as a primary geotechnical concern (due to the uniquely large cantilever loading imposed by the west side of the residence). Are the recommended skin friction values (1600 psf for uplift loading, 2000 psf for downward loads) and minimum 1 foot of embedment appropriate considering the degree of fracturing present in the bedrock and the apparent absence of an acceptable ASTM laboratory shear strength test to substantiate these values?

5. The consultant should provide a discussion regarding whether shoring (soldier piles and lagging) will be left in place or removed. If it is to be removed, provide recommendations to prevent settlement of adjacent materials (including impacts to the adjacent residence) during removal of lagging and soldier piles.

6. Section 5.8 of the report states: "*The top lift of trench backfill under vehicle pavements should be compacted to the requirements given in report section 5.3 for vehicle pavement subgrades*" however, report section 5.3 doesn't present compaction requirements for vehicle pavement subgrades. Provide additional recommendations as appropriate.

Civil/Structural Plan Comments -- Review of the Site Plan, Sections, Residence Profile, and Shoring Detail (sheets 1 through 5) by Shoreline Engineering, dated January 2012 generated the following comments:

1. Note 4 on the Shoring Detail (sheets 4 and 5) should reference the new subsurface data and updated cross-sections by the consultants, including CHG boring HA-9 on the Shoring Detail at SL-1 and GSI boring B-5 on the Shoring Detail at SL-2. Sandstone bedrock may actually be shallower than that shown on these details. This, in turn, may reduce to some degree the depth and magnitude of drilling required for shoring installation.

2. The plans by Shoreline Engineering are unclear as to whether the driveway is to be supported on engineered fill (bounded by retaining walls on three sides), or will it be a structural slab spanning between Studio Drive and the residence? The GSI report (page 7) states: "*It is also anticipated that the driveway will span between the residence and Studio Drive. A retaining wall will likely be required adjacent to Studio Drive*", but then states, "*If compacted select soils are to be placed for the driveway all existing fills should be removed to competent materials.*" This question impacts grading quantity estimates, import soil quantities, and related environmental impact considerations, and should be resolved by the project designers.

LIMITATIONS

This supplemental geotechnical peer review has been performed to provide technical advice to SWCA Environmental Consultants pursuant to its preparation of the Environmental Impact Report for the referenced project. Our services consist of professional opinions and recommendations made in accordance with generally accepted engineering geology and geotechnical engineering principles and practices. No warranty, expressed or implied, or merchantability of fitness, is made or intended in connection with our work, by the proposal for consulting or other services, or by the furnishing of oral or written reports or findings.

Any recommendations presented in this report are contingent upon our firm being retained to review and comment on the Draft Environmental Impact Report.

Respectfully submitted,

COTTON, SHIRES AND ASSOCIATES, INC.

Michael B. Phipps
Principal Engineering Geologist
CEG 1832

David T. Schrier
Principal Geotechnical Engineer
GE 2334

MP:DTS:st

COTTON, SHIRES AND ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

October 31, 2012
SC0099

Ms. Shawna Scott
SWCA ENVIRONMENTAL CONSULTANTS
1422 Monterey Street, Suite C200
San Luis Obispo, California 93401

SUBJECT: **Second Supplemental Geotechnical Peer Review for Environmental Impact Report Preparation**
RE: Loperena Minor Use Permit/Coastal Development Permit
Studio Drive, Cayucos, San Luis Obispo County, CA

Dear Ms. Scott:

In accordance with your request, Cotton, Shires and Associates, Inc. (CSA) has performed a second supplemental geotechnical peer review of materials submitted for the Loperena Minor Use Permit/Coastal Development Permit. We received and reviewed the following documents:

- Cleath-Harris Geologists, Inc., 2012, Update #2 to Engineering Geology Reports for the Proposed Loperena Residence, Lot 41, Studio Drive, Cayucos, California, dated September 19, 2012;

- GSI Soils Inc., 2012, Response to Supplemental Geotechnical Peer Review, Loperena Residence, Lot 41, Studio Drive, Cayucos, California, dated October 1, 2012; and

- Shoreline Engineering, 2012, Re: Loperena, County of San Luis Obispo, Responses to Supplemental Geotechnical Peer Review for EIR Preparation, 8/21/12, dated September 20, 2012; and Shoring Detail Sheets 4 and 5 dated January 2012 (no revision date provided).

The documents referenced above were peer-reviewed from engineering geologic and geotechnical engineering perspectives, with reference to our previous peer review comments dated August 21, 2012 and technical report dated May 31, 2011 The purpose of this second supplemental review was to determine whether the previous geotechnical peer review comments have been satisfactorily addressed such that the additional project

information and recommendations from the consultants can be incorporated into the project EIR.

DISCUSSION

Based upon our review of the referenced materials submitted by the project team, the comments presented in CSA's August 21, 2012 supplemental peer review have been addressed to our satisfaction. We have no further peer review comments. We recommend that the updated findings, conclusions, information and recommendations presented by the consultants be incorporated into the project environmental impact report as appropriate.

LIMITATIONS

This supplemental geotechnical peer review has been performed to provide technical advice to SWCA Environmental Consultants pursuant to its preparation of the Environmental Impact Report for the referenced project. Our services consist of professional opinions and recommendations made in accordance with generally accepted engineering geology and geotechnical engineering principles and practices. No warranty, expressed or implied, or merchantability of fitness, is made or intended in connection with our work, by the proposal for consulting or other services, or by the furnishing of oral or written reports or findings.

Any recommendations presented in this report are contingent upon our firm being retained to review and comment on the Draft Environmental Impact Report.

Respectfully submitted,

COTTON, SHIRES AND ASSOCIATES, INC.

Michael B. Phipps
Principal Engineering Geologist
CEG 1832

David T. Schrier
Principal Geotechnical Engineer
GE 2334

MP:DTS:st

COTTON, SHIRES AND ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

May 17, 2013
SC0099

VIA EMAIL: sscott@swca.com

Ms. Shawna Scott
SWCA ENVIRONMENTAL CONSULTANTS
1422 Monterey Street, Suite C200
San Luis Obispo, California 93401

SUBJECT: **Additional Geotechnical and Coastal Engineering Review and Response to Technical Comments**
RE: Loperena Minor Use Permit/Coastal Development Permit
Studio Drive, Cayucos, San Luis Obispo County, CA

REFERENCES: Cotton, Shires and Associates, Inc., October 31, 2012, Supplemental Geotechnical Peer Review (No. 2) for Environmental Impact Report Preparation, Loperena Minor Use Permit/Coastal Development Permit, Studio Drive, Cayucos, San Luis Obispo County, CA

Cotton, Shires and Associates, Inc., August 21, 2012, Supplemental Geotechnical Peer Review for Environmental Impact Report Preparation, Loperena Minor Use Permit/Coastal Development Permit, Studio Drive, Cayucos, San Luis Obispo County, CA

Cotton, Shires and Associates, Inc., May 31, 2011, Technical Report: Geotechnical and Coastal Hazards Review, Loperena Minor Use Permit/Coastal Development Permit, APN 064-253-07, Studio Drive, Cayucos, San Luis Obispo County, California

GeoSoils Inc., March 14, 2011, Discussion of Coastal Hazards and Wave Runup, Northwest and Immediately Adjacent to 2612 Studio Drive (APN 064-253-07), Cayucos, San Luis Obispo County, California

Dear Ms. Scott:

In accordance with your request, Cotton, Shires and Associates, Inc. (CSA) is pleased to provide the following letter containing a summary of our additional geotechnical and coastal engineering review and our response to technical comments regarding the above

referenced project. We understand that the County of San Luis Obispo received some public comment with regard to the project, both before and after our review of the project, that they have requested be addressed in the Draft EIR. Specifically, those technical comments were presented in the following documents:

- Haro, Kasunich and Associates, Inc., November 12, 2007, Review of Residential Development On Coastal Bluff and Supporting Geologic and Geotechnical Reports Prepared for Development, Loperena Property, APN 064-253-007, Lot 41, Studio Drive, Cayucos, San Luis Obispo County, California; *contained as Exhibit A in* Sinsheimer Juhnke Lebens & McIvor, LLP, April 16, 2009, Letter Re: Loperena MUP/CDP: DRC2005-00216 – Attachment to Request for Review of Proposed Amended Mitigated Negative Declaration and Notice of Determination;

- Earth Design, April 16, 2009, Letter RE: April 2, 2009 Amended Initial Study-Loperena Minor Use Permit; *contained as Exhibit C in* Sinsheimer Juhnke Lebens & McIvor, LLP, April 16, 2009, Letter Re: Loperena MUP/CDP: DRC2005-00216 – Attachment to Request for Review of Proposed Amended Mitigated Negative Declaration and Notice of Determination; and

- Haro, Kasunich and Associates, Inc., March 13, 2012, Review of Additional Documents, Residential Development on Coastal Bluff, Loperena Property, APN 064-253-007, Lot 41, Studio Drive, Cayucos, San Luis Obispo County, California; *enclosure in* Sinsheimer Juhnke McIvor & Stroh, LLP, Letter Re: Loperena Environmental Impact Report, Studio Drive, Cayucos, APN 064-253-007, ED06-317, DRC 2005-00216.

CSA and our subcontracted coastal engineering consultant, Mr. David Skelly, PE, of GeoSoils, Inc. have reviewed the technical comments in the above referenced materials. As indicated in our previously submitted documents referenced above, we have performed significant independent peer review of this project and that review work largely serves as the basis for our response to the public comment with regard to geological/geotechnical engineering items. In addition, David Skelly has prepared a supplemental report further addressing coastal hazards and wave runup issues (attached).

Based upon our review of the public comment, this letter and the attached supplemental report are intended to further address the following key items:

- Coastal bluff determination/interpretation;
- Resolution of other geotechnical concerns;
- Tsunami inundation potential;

- Clarification of wave runup and future sea level rise estimates using California Ocean Protection Council estimates (per California Coastal Commission);
- Additional input regarding potential sea level rise and coastal erosion potential; and
- Discussion of wave runup reflection potential.

Coastal Bluff Determination/Interpretation

A significant underlying basis for the code and policy compliance issues cited by Haro, Kasunich and Associates, Inc. (hereafter, HKA) in their 2007 and 2012 reviews, is their opinion that the Loperena project site is located on a coastal bluff. This opinion appears to be based on their interpretation of coastal bluff termini presented in Figure 1, "Coastal Bluff Line", of their 2007 and 2012 correspondence. CSA performed a detailed analysis of this topic as presented in Section 3.4 Coastal Bluff Interpretation of our May 31, 2011 technical report. This work included review of historic aerial photographs dating back to 1937 (see Section 2.2 Development History), and included consultation with the California Coastal Commission staff, review of their guidelines for CCR Title 14, Section 13577, and conducting our own analysis to determine the terminus of the coastal bluff. We found, based on our interpretation and application of the CCC guidelines for CCR Title 14, Section 13577, that the Loperena property is not located on a coastal bluff.

We have reviewed the "Coastal Bluff Line" interpretation presented in HKA's Figure 1 (2007, 2012). HKA's interpretation, presented on a 2007-era aerial photograph, is inappropriate because their "Seaward Facing Bluff Line" drawn through the Loperena property and extending hundreds of feet north, is drawn along an artificial fill slope constructed pre-1963 for the extension of Studio Drive. Furthermore, their "Inland Bluff" is drawn along an artificial fill slope constructed across the alluvial valley of Old Creek pre-1963 for the expansion and realignment of Old Cabrillo Highway (CA-1). We believe it is inappropriate to consider that manmade features such as artificial fill prisms graded for roadway developments comprise "bluffs". An analysis to determine the terminus of a natural feature, such as a coastal bluff, should not be based upon manmade topographic features. Furthermore, it should be clear upon review of the inland bluff feature delineated on CSA Figure 5 (2011), that these artificial fill prisms constructed circa 1960 were not present in 1937. HKA (2007, page 1) stated that: "Where a coastal bluff curves landward to become a canyon bluff, the terminus of the coastal bluff line is the location where the seaward facing portion of the bluff turns and faces inland." We find this statement to be consistent with our coastal bluff interpretation, which is based in part on an inland bluff location that is now concealed both beneath and northeast of the property. Following the Coastal Commission's guidelines, we determined through analysis that the coastal bluff terminates immediately south of the Loperena property.

Coastal Erosion

The HKA 2007 and 2012 letters cite USGS Open-File Report 2007-1133 and state that it indicates that cliff retreat rates in the vicinity of the proposed Loperena residence are 0.5 to 0.9 feet per year. HKA opines that the site is subject to coastal erosion. Review of cliff retreat rates vs. distance along shore presented in Figure 27 of USGS OFR 2007-1133 indicates that, at a minimum, the scale at which the cliff retreat data is presented does not allow for interpretation of a cliff retreat rate over the extremely narrow (25 feet) width of the site. Furthermore, we determined through analysis that the project site is not located on a coastal bluff; rather, the property is situated atop a bedrock remnant of the inland bluff adjacent to the mouth of Old Creek. The property is clearly set back significantly landward of the general trend of the coastal bluff, which terminates immediately southeast of the subject property (see Figure 6, CSA, 2011). In terms of the cliff retreat rate cited, this was likely determined at properties south of the subject property that actually are situated atop a coastal bluff. The topic of coastal erosion hazard, including the effects of sea level rise was also addressed by our coastal engineering consultant, David Skelly (see GeoSoils, Inc., 2011). The sea level rise scenario was revisited in a supplemental report (GeoSoils, 2013) attached hereto, and resulted in the same conclusion as the 2011 report, that the proposed residence should be reasonably safe from coastal hazards over its economic life.

Other Geotechnical/Geologic Hazard Concerns

Other geotechnical or geologic hazard concerns cited in HKA (2007, 2012) and Earth Design (2009) correspondence include the topics of groundwater, seismicity, liquefaction, expansive soil, erosion, slope instability and sedimentation. Technical discussion of these topics was presented and/or summarized in our hazards review (CSA, 2011). We also performed three independent peer reviews (CSA, 2011; 2012; 2012) of the initial and addendum engineering geologic and geotechnical engineering reports and engineered plans (grading, drainage and shoring) submitted by the applicant's consultants. Peer review comments were addressed to our satisfaction and we remain of the opinion that the information, findings, conclusions and recommendations presented by the applicant's consultants are appropriate for incorporation into the project environmental impact report and the project design.

Coastal Hazard Concerns

As you are aware, a study addressing coastal hazards and wave runup was prepared by our subconsultant coastal engineer, David Skelly, PE, of GeoSoils, Inc. (GeoSoils, 2011) and was appended to our May 2011 Technical Report. A supplemental report specifically addressing extreme wave runup and wave runup reflection under future sea level rise scenarios, and providing additional comment with regard to tsunami impacts on the

development, was recently prepared (GeoSoils, Inc., 2013) and is attached to this letter. The supplemental report incorporates recent sea level rise predictions and presents an updated wave runup and overtopping analysis based upon CSA's Engineering Geologic Cross-Section 1-1'. In the supplemental report, David Skelly provides conclusions consistent with his 2011 study.

Limitations

This letter was prepared to provide technical advice to SWCA Environmental Consultants pursuant to its preparation of the Environmental Impact Report for the referenced project. Our services consist of professional opinions and recommendations made in accordance with generally accepted engineering geology and geotechnical engineering principles and practices. No warranty, expressed or implied, or merchantability of fitness, is made or intended in connection with our work, by the proposal for consulting or other services, or by the furnishing of oral or written reports or findings.

We trust that this letter provides you with the information that you need at this time. Should you have any questions, please do not hesitate to contact our office.

Very truly yours,

COTTON, SHIRES AND ASSOCIATES, INC.

Michael B. Phipps
Principal Engineering Geologist
PG 5748, CEG 1832

Patrick O. Shires
Senior Principal Geotechnical Engineer
GE 770

MP:POS:st

Attachment: GeoSoils, Inc., 2013, Supplemental Discussion of Coastal Hazards and Wave Runup, APN 064-253-07, Cayucos, San Luis Obispo County, CA

GeoSoils Inc.

April 10, 2013

Mr. Michael Phipps
Cotton, Shires and Associates, Inc.
550 St. Charles Drive, Suite 108
Thousand Oaks, CA 91360-3995

Subject: Supplemental Discussion of Coastal Hazards and Wave Runup, APN 064-253-07, Cayucos, San Luis Obispo County, California

Dear Mr. Phipps:

At your request and authorization, GeoSoils Inc. (GSI) is pleased to present the following supplemental report concerning the coastal hazards and wave runup associated with the proposed residential structure development at subject site. The report is a supplement to our March 14, 2011 coastal hazard and wave runup study, and unless specifically superceded herein, the conclusions of that report remain valid and pertinent. This additional discussion is based upon additional information contained in the Cotton, Shires and Associates Inc. (CSA), Geotechnical and Coastal Hazards Review and addresses coastal hazard issues raised in public comments received on the project as well as from the County's review of the Administrative Draft EIR for the project. The public comment received raised concerns about future sea level rise, wave runup, wave runup reflection, and the impact of a tsunami on the project. This report specifically addresses extreme wave runup and wave runup reflection under future sea level rise scenarios, and the potential for tsunami impacts on the development.

SITE DESCRIPTION

The subject site is an irregularly shaped parcel at the northern end of Studio Drive. Site elevations vary from about +10 feet NAVD88 at the beach area to about elevation + 30 feet NAVD88 at Studio Drive. The majority of the parcel is at or above elevation +20 feet NAVD88. The site is fronted by a bedrock outcropping (Graywacke Sandstone) from about elevation +17 feet NAVD88 to the beach at about elevation +10 feet NAVD88. This irregular bedrock serves as a form of natural shore protection. Photograph 1 is an aerial photograph taken in September 25, 2010 downloaded with permission from the California Coastal Records Project web site (http://www.californiacoastline.org/).

Photograph 1. Subject site and adjacent development and shoreline September 25, 2010.

SEA LEVEL RISE

The City of Newport Beach contracted Everest International Consultants, Inc. (EICI) to produce an assessment report on the Balboa Island seawall(s) (EICI, 2011). This report provides a comprehensive discussion of future sea level rise estimates on a national, state and local (Newport Beach) level. The report analysis is applicable to the central coast of California and the subject Cayucos site. In addition to the EICI report, the U.S. Army Corps of Engineers has provided a guideline for incorporating sea level change in civil works projects (U.S. Army Corps of Engineers, 2009). The Corps of Engineers is a leader in setting the standard for coastal engineering practice. The 2009 guideline provides a high, an intermediate, and a low sea level rise (above current sea level) estimates.

Figure 2, from the EICI assessment report, provides a comparison of the sea level rise (SLR) projections of the Corps of Engineers, the California Ocean Protection Council (COPC), and the California Coastal Conservancy. It is clear that while there is some agreement over the next 20 years, beyond 20 years from now there is little agreement on SLR projections. As depicted in Figure 2, the California Ocean Protection Council (COPC) has adopted the most aggressive sea level rise prediction to date. The proposed residential structure has an expected life of 75 to 100 years. To be reasonably conservative and for analysis purposes we will use 2.5 feet, which is about in the middle of the range in Figure 2. The highest recorded water elevation in the vicinity is +7.57 feet NAVD88 at Port San Luis. This elevation includes all short-term oceanographic effects on sea level but not the long-term sea level rise prediction. If 2.5 feet is added to this +7.57 feet NAVD88 elevation a future design maximum sea level of+10.1 feet NAVD88 is determined.

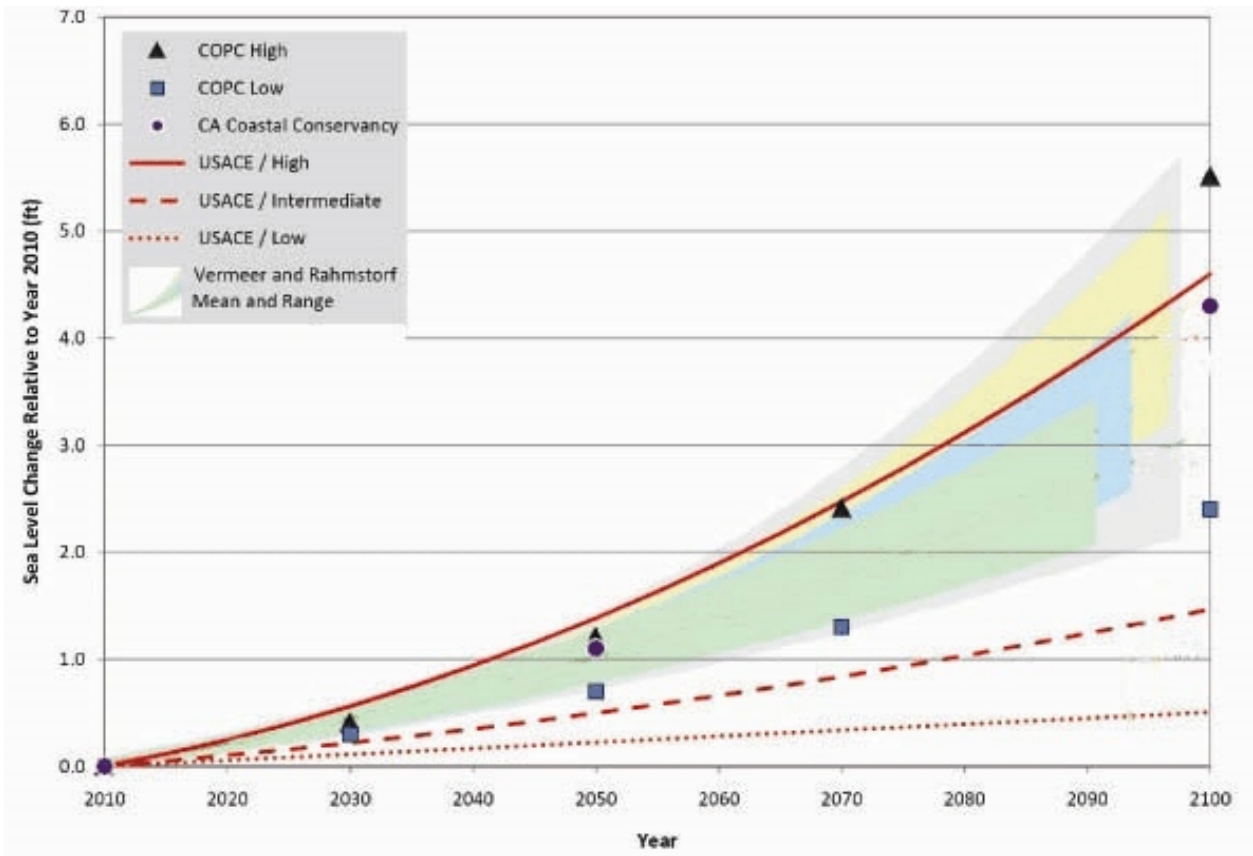


Figure 2. Sea Level Rise range of estimates from Everest International Consultants, Inc.

GeoSoils Inc.

UPDATED WAVE RUNUP & OVERTOPPING

Based upon the new sea level rise predictions GSI's review of CSA's Engineering Geologic Cross-Section 1-1', and additional historical aerial photographs the following are the updated design oceanographic conditions expected over the life of the development.

Design still water elevation	**+10.1 feet NAVD88**
Design scour at base of slope	**+ 3.1 feet NAVD88**
Water depth at toe	**7.0 feet**
Design wave height at toe and period	**5.5 feet and 18 seconds**
Slope	**1/ 2.2**
Nearshore slope	**1/ 14**

Using the updated parameters in the ACES analysis yields the following results.

```
AUTOMATED COASTAL ENGINEERING SYSTEM ... Version 1.02     4/ 9/2013    7:31
Project:     WAVE RUNUP SUPPLEMENT 2612 STUDIO DRIVE, CAYUCOS
_____________________________________________________________________________
            WAVE RUNUP AND OVERTOPPING ON IMPERMEABLE STRUCTURES
              Item                        Unit        Value
Wave Height at Toe            Hi:   ft              5.500       Rough Slope
Wave Period                    T:   sec            18.000        Runup and
COTAN of Nearshore Slope                           50.000       Overtopping
Water Depth at Toe            ds:   ft              7.000
COTAN of Structure Slope                            2.200
Structure Height Above Toe    hs:   ft             14.000
Rough Slope Coefficient        a:                   0.956
Rough Slope Coefficient        b:                   0.398
Deepwater Wave Height         H0:   ft              3.118
Relative Height          (ds/H0):                   2.245
Wave Steepness         (H0/gT^2):                   0.299E-03
Wave Runup                     R:   ft             10.021
Onshore Wind Velocity          U:   ft/sec         16.878
Overtopping Coefficient    Alpha:                   0.500E-01
Overtopping Coefficient   Qstar0:                   0.130
Overtopping Rate               Q:   ft^3/s-ft       0.271
```

The rock outcropping at its average elevation of +17 feet NAVD88 can be overtopped by the design wave at a rate of about 0.27 ft^3/sec-ft. The original coastal hazard analysis produced an overtopping rate of about 1 ft^3/sec-ft. Based upon this updated wave runup

and overtopping, using a higher sea level rise and a higher scour elevation, our original analysis is still valid and possibly conservative.

Using the following empirical formulas provided by the USACOE the height of the water at the top of the slope (+17 feet NAVD88) h_1 and the velocity of the water at the top v_c of the slope can be calculated. .

$$q = 0.8927\sqrt{g}, h_1^{3/2}$$

$$v_c = (1.64)\sqrt{\frac{2}{3}gh_1}$$

For the original report overtopping rate of 1 ft^3/sec-ft calculated by ACES the water height is 0.8 feet and the velocity is ~6 feet per second. This low height of water and relatively low velocity will not impact the development behind it. In addition, the lower overtopping rate calculated herein and resulting low water height and low velocity will also have no impact on the adjacent property improvements. Finally, both the water height and velocity decay, as the water travels, within a few seconds.

TSUNAMI

The original coastal hazard report provides a discussion of potential tsunami impacts on the site. This analysis remains valid and pertinent. In addition to our site specific analysis, the State of California, County of San Luis Obispo produced a regional tsunami inundation map. This map is not intended to be a site specific analysis and its primary purpose is for coastal evacuation planning only. The proposed development will be constructed to current building codes which as determined above will adequately protect the development from wave hazards, including tsunami type waves.

GeoSoils Inc.

CONCLUSIONS

As previously concluded in our original coastal hazard study, the proposed residence is reasonably safe from coastal hazards over its economic life. There are no recommendations necessary to mitigate potential coastal hazards. New shore protection will not be required to protect the proposed residence over the next 100 years. The proposed residence will neither create nor contribute to erosion, geologic instability, or destruction of the site or adjacent area.

LIMITATIONS

Coastal engineering is characterized by uncertainty. Professional judgments presented herein are based partly on our evaluation of the technical information gathered, partly on our understanding of the proposed construction, and partly on our general experience. Our engineering work and judgments have been prepared in accordance with current accepted standards of engineering practice; we do not guarantee the performance of the project in any respect. This warranty is in lieu of all other warranties express or implied.

Respectfully Submitted,

GeoSoils Inc.

David W. Skelly MS, PE

REFERENCES

Environmental Protection Agency http://epa.gov/climatechange/science/futureslc

California Emergency Management Agency, California Geological Survey, University of Southern California, 2009, "Tsunami Inundation Map for Emergency Planning, Cayucos Quadrangle, State of California, County of San Luis Obispo," Scale 1:24,000, dated July

Everest International Consultants, Inc., 2011, “Assessment of seawall structure integrity and potential for seawall over-topping for Balboa Island and Little Balboa Island, main report,” No Project No., dated April 21.

NOAA, 2011, Web Site, Maps http://anchor.ncd.noaa.gov/states/ca.htm Tidal Datums http://www.opsd.nos.noaa.gov/cgi-bin/websql/ftp/query_new.pl

U.S. Army Corps of Engineers, 2009, Water resources policies and authorities incorporating sea-level change considerations in civil works programs.

Cleath-Harris Geologists, Inc.
11545 Los Osos Valley Road, Suite C-3
San Luis Obispo, California 93405
(805) 543-1413

June 25, 2012

Bruce Elster
Shoreline Engineering
505 Harbor Street
Morro Bay, California 93442

SUBJECT: Updates to Engineering Geology Reports for the Proposed Loperena Residence, Lot 41, Studio Drive, Cayucos, California

Dear Mr. Elster:

In response to the Technical Report: Geotechnical and Coastal Hazards Review for the Loperena Minor Use Permit/Coastal Development Permit by Cotton, Shires and Associates (CSA), May, 2011, Cleath-Harris Geologists (CHG) is providing updates to the engineering geology reports of 2006 and 2007 by Cleath & Associates.

Included in Section 1.6 of the CSA report are peer review comments on previous project reports and memoranda relating to both engineering geology and geotechnical studies for the project.

Section 1.6.1 includes the following engineering geology comments (italics) by CSA. Our responses and documentation of additional site investigation follow each item.

1. *Update seismic design parameters to the current codes (IBC 2009/CBC 2010).*

   The seismic design parameters were subsequently provided by GSI Soils in the "Updated Geotechnical Investigation", December 27, 2011

2. *Provide references and information regarding recent activity attributed to the Oceanic fault.*

   The Oceanic fault is the nearest fault to the site with documented evidence of recent seismic activity. The activity includes the December 2003, 6.5 magnitude San Simeon Earthquake attributed to the Oceanic fault by the U.S. Geological Survey. The San Simeon earthquake was caused by reverse faulting, and was centered approximately 22 miles north of the project site. (USGS Earthquake Hazards Program, 2003). A more recent 4.4 magnitude earthquake on June 20, 2009, occurred along the Oceanic fault according to the Geological Survey. No injuries were reported from the quake.

   The Oceanic fault is the northwestern segment of the West Huasna/Oceanic fault zone that trends north-northwest for approximately 100 kilometers extending from approximately the Santa Maria River on the south to San Simeon on the north. The Oceanic fault segment continues northward from near San Luis Obispo to San Simeon (County of San Luis Obispo and Cities Safety Element Update). A slip rate in terms of distance of slip per year has not

been established in the California Geological Survey "California Fault Parameters for the National Seismic Hazard Maps and Working Group on California Earthquake Probabilities" (Wills, 2007).

3. *Provide a complete geologic map of the property illustrating the distribution of earth units at the ground surface, including limits of undocumented fill and other surficial deposits.*

   A site geologic map was generated in the 2011 CSA report showing alignments of two new geologic cross sections by CSA as an overlay on a base map by Shoreline Engineering that illustrates locations of proposed subsurface structures. A geologic map by CHG showing locations of all subsurface exploration related to the project and all geologic cross section alignments is included herein in Figure 1.

4. *Exploration is confined to the north half of the property. There is a lack of subsurface exploration data: 1) near/along the southerly property line where conditions are most critical due to excavations required for basement retaining walls and close proximity of existing development next door; and 2) through the fill slope supporting Studio Drive. Additional subsurface exploration is warranted.*

   CSA completed the longitudinal Cross Section 1–1' through existing exploration borings and test pits in the north half of the property and Cross Section 2-2' extending cross section B previously completed by CHG (CSA, 2011). GSI Soils subsequently advanced Borings 4 and 5 near the southern property line and extended Cross Sections A and B (GSI, 2011). Cross Section B (GSI, 2011) illustrates the shoring system and the location of engineered fill.

   Because no additional subsurface exploration was performed in the critical area near the driveway in the southern portion of the property, and near the westerly area of the proposed residential basement, CHG advanced four hand-auger borings and completed a longitudinal cross section near the southern property line (Figure 2, Cross Section D-D'). The existing Cross Section C-C' was updated (Figure 3) to include subsurface information obtained from boring HA-10.

   The four borings completed by CHG along the southern property line were advanced using hand-auger equipment. Each boring was drilled until refusal occurred. Fill materials were observed to five feet depth in boring HA-9; to three feet depth in borings HA-10 and HA-10A; and to four and one half feet depth in boring HA-11. Fill materials consisted of silt, sand and gravel in boring HA-9, and mostly clay in the three remaining borings. Beneath the fill material, clayey gray to dark brown angular sandstone clasts were observed to the depth of refusal. These sandstones were interpreted to make up a thin weathered and broken veneer at the top of resistant sandstone bedrock. The depth of refusal was interpreted to be the top of or very near the top of the sandstone bedrock. Lithologic logs for each of the four borings are attached.

5. *The cited wave run-up and tsunami elevations are based upon regional studies from 1978 to 1980. A site-specific analysis should be performed to best define the hazard(s).*

   A discussion of coastal hazards including shoreline erosion, wave runup, and coastal flooding northwest and immediately adjacent to the project site was provided by David W. Skelly of GeoSoils, Inc., March 14, 2011.

Please contact our office if you have any questions or comments.

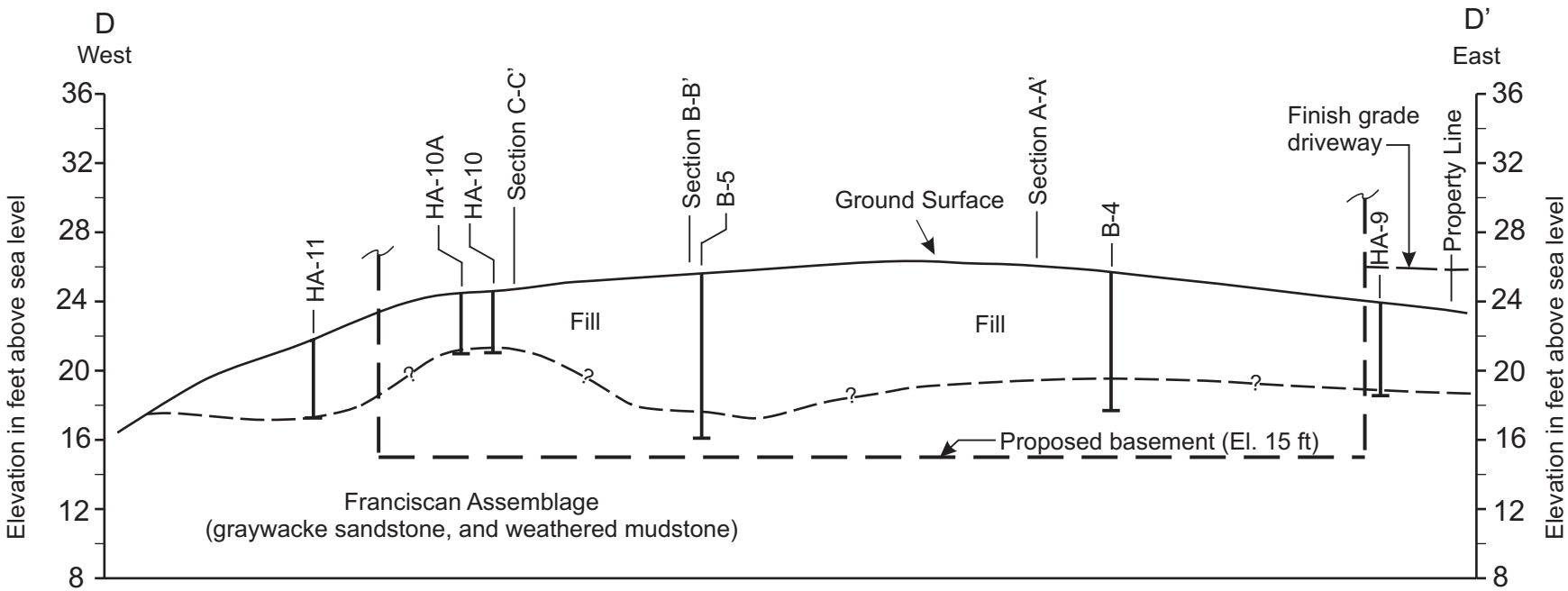
Sincerely,
CLEATH-HARRIS GEOLOGISTS, INC.

David R. Williams
Associate Geologist

Timothy S. Cleath
Certified Engineering Geologist #1102

Figure 1
Geologic Map
Loperena Property, Studio Drive
Cayucos, California
May 18, 2012

Cleath-Harris Geologists



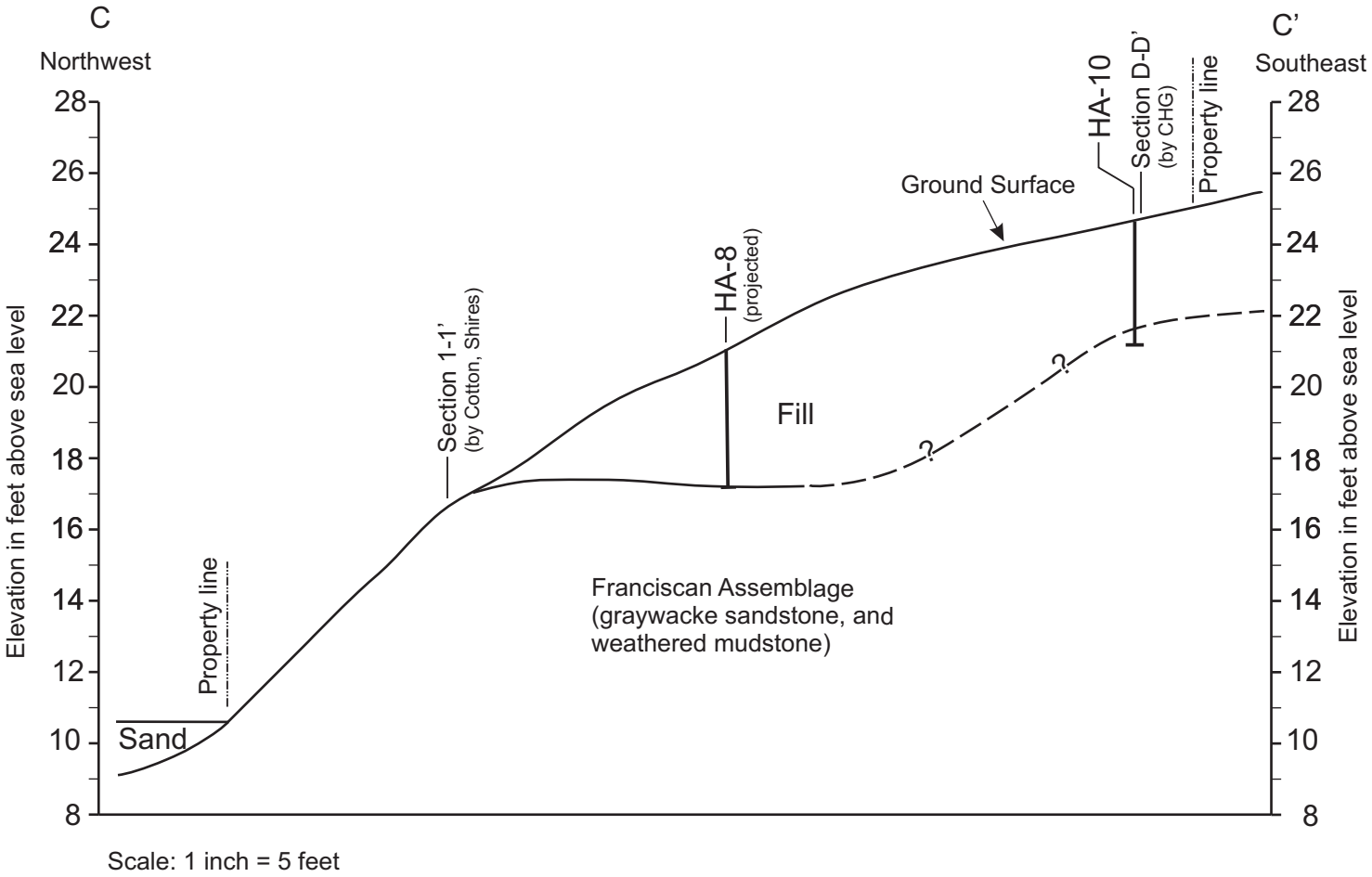
Scale: 1 inch = 10 feet

Explanation

HA-11	Hand auger boring location (by CHG)
B-5	Power auger boring location (by GSI)
-?	Geologic contact, queried and dashed where inferred

Figure 2
Cross Sections D-D'
Loperena Property, Studio Drive
Cayucos, California
May 18, 2012

Cleath-Harris Geologists



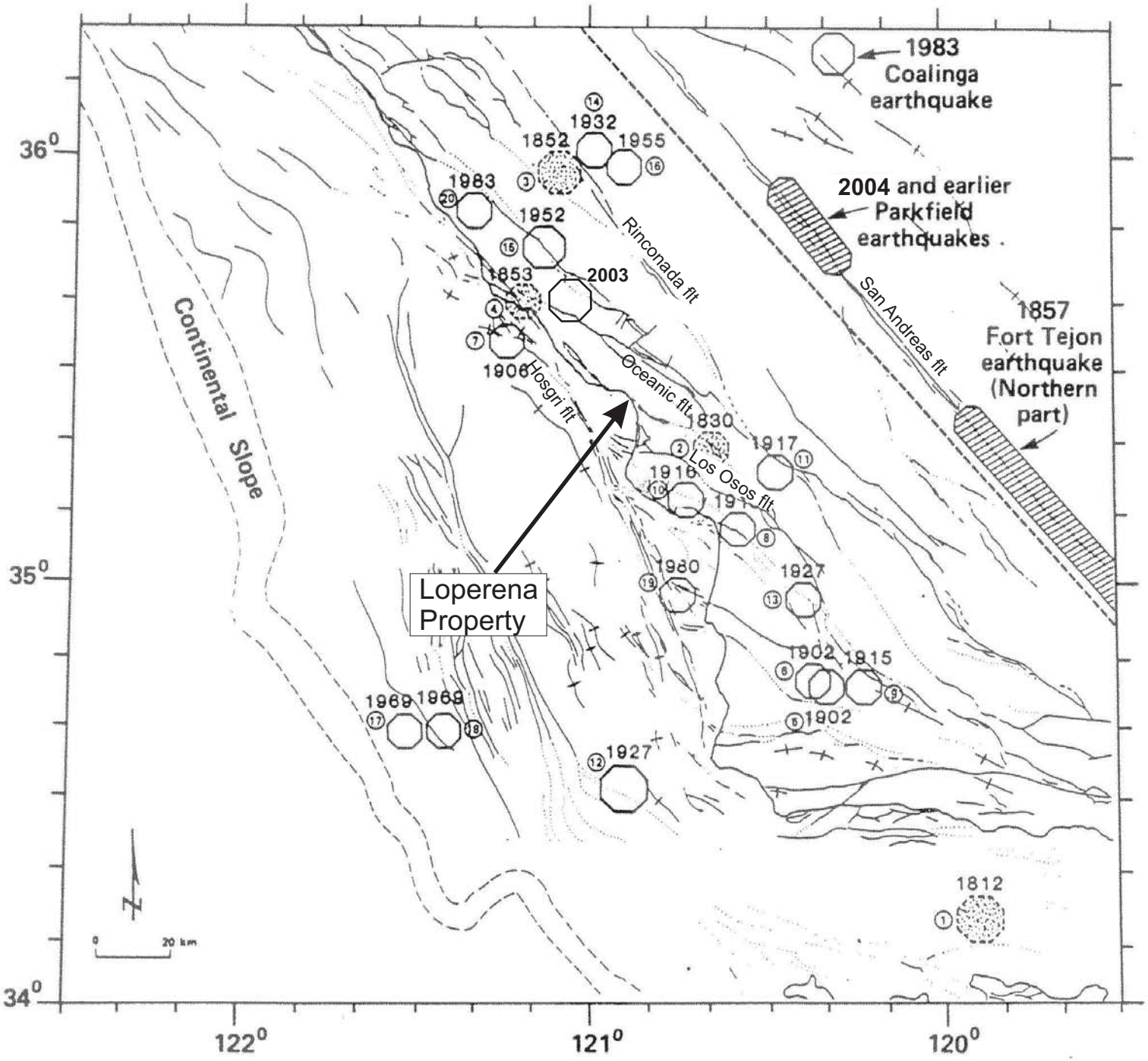
Explanation

HA-10 Hand auger boring location

? Geologic contact, queried and dashed where inferred

Figure 3
Revised Cross Section C-C'
Loperena Property, Studio Drive
Cayucos, California
May 18, 2012

Cleath-Harris Geologists



EXPLANATION

Earthquake epicenter:

1812 – Year of event

① Event number

Event location that is poorly constrained

Event location that is within 20 kilometers and generally within 10 kilometers

Magnitude

5.0 – 5.9

6.0 – 6.9

7.0 – 7.9

Base Map: Diablo Canyon Power Plant
Long Term Seismic Program - Final Report
Pacific Gas & Electric, July 1988

0 40 80

Scale: 1 inch = 40 kilometers

Figure 4

Earthquake Epicenter Map
Loperena Property

Cleath-Harris Geologists

Attachments

Lithology Logs
of 2012 Hand–Auger Borings

Boring Logs
Loperena Property, Studio Drive, Cayucos, California

Date of borings: May 10, 2012
Geologists: Williams, Pfeifle; Cleath-Harris Geologists.
Drilling Method: hand auger

Boring HA-9
Elevation: 23.7 ft above sea level (from topographic map)
Total depth: 5.5 feet
Depth to top and bottom in feet

Top	Bottom	Thickness	Description
0	1.5	1.5	**FILL;** silty sand with gravel; dark brown; fine grained sand; moist.
1.5	3	1.5	**FILL;** sand with silt and gravel; dark brown; fine to medium grained sand; gravel to 3”, subrounded; fine roots; moist.
3	5	1	**FILL;** gravely sand with silt; dark brown; fine to medium grained sand; gravel to 3”, angular to rounded; moist.
5	5.5	0.5	**Sandstone;** greywacke sandstone fragments; gray to dark brown; moist.

Refusal at 5.5 feet depth.

Boring Logs
Loperena Property, Studio Drive, Cayucos, California

Date of borings: May 10, 2012
Geologists: Williams, Pfeifle; Cleath-Harris Geologists.
Drilling Method: hand auger

Boring HA-10
Elevation: 24.2 ft above sea level (from topographic map)
Total depth: 3.5 feet
Depth to top and bottom in feet

Top	Bottom	Thickness	Description
0	1	1	**FILL;** sandy clay; dark brown, variegated dark red brown; soft; fine to medium sand; moist.
1	2	1	**FILL;** sandy clay with gravel; dark brown; soft; fine to medium sand; gravel to 2”, roots and galvanized wire; moist.
2	3	1	**FILL;** clay with sand and gravel; stiff; fine sand; greywacke gravel to 1”, angular; moist.
3	3.5	0.5	**Sandstone;** greywacke sandstone fragments; gray to dark brown; moist.

Refusal at 3.5 feet depth.

Boring Logs
Loperena Property, Studio Drive, Cayucos, California

Date of borings: May 10, 2012
Geologists: Williams, Pfeifle, Cleath-Harris Geologists.
Drilling Method: hand auger

Boring HA-10A
Elevation: 24.2 ft above sea level (from topographic map)
Total depth: 3.5 feet
Depth to top and bottom in feet

Top	Bottom	Thickness	Description
0	1	1	**FILL;** sandy clay; dark brown, variegated dark red brown; soft; fine to medium sand; moist.
1	2	1	**FILL;** sandy clay with gravel; dark brown; soft; fine to medium sand; gravel to 2”, roots and galvanized wire; moist.
2	3	1.5	**FILL;** clay with sand and gravel; stiff; fine sand; greywacke gravel to 1”, angular; moist.
3	3.5	0.5	**Sandstone:** greywacke sandstone fragments; gray to dark brown; moist.

Refusal at 3.5 feet depth.

Boring Logs
Loperena Property, Studio Drive, Cayucos, California

Date of borings: May 10, 2012
Geologists: Williams, Pfeifle, Cleath-Harris Geologists.
Drilling Method: hand auger

Boring HA-11
Elevation: 21.8 ft above sea level (from topographic map)
Total depth: 4.5 feet
Depth to top and bottom in feet

Top	Bottom	Thickness	Description
0	2	2	**FILL;** clay with sand; variegated redish brown to dark brown; soft; fine sand; moist.
2	3	1	**FILL;** clayey sand; dark brown; medium consistency; fine sand; roots; moist.
3	3.5	0.5	**FILL;** sandy clay with rock fragments; dark brown to black; soft consistency; angular; moist.
3.5	4	0.5	**FILL;** clay; yellowish brown; stiff consistency; moist.
4	4.5	0.5	**FILL;** sandy clay with gravel; yellowish brown; fine to medium grained sand; greywacke sandstone fragments to 1”; refusal on flat surface at 4.5’.

Refusal at 4.5 feet depth.

Cleath-Harris Geologists, Inc.
11545 Los Osos Valley Road, Suite C-3
San Luis Obispo, California 93405
(805) 543-1413

September 19, 2012

Bruce Elster
Shoreline Engineering
505 Harbor Street
Morro Bay, California 93442

SUBJECT: Update #2 to Engineering Geology Reports for the Proposed Loperena Residence, Lot 41, Studio Drive, Cayucos, California

Dear Mr. Elster:

In response to the Technical Report: Geotechnical and Coastal Hazards Review for the Loperena Minor Use Permit/Coastal Development Permit by Cotton, Shires and Associates (CSA), May, 2011, Cleath-Harris Geologists (CHG) is providing this second update to the engineering geology reports of 2006 and 2007 by Cleath & Associates. The first update by CHG was submitted June 25, 2012. This second update revises Geologic Cross Section B-B' by CHG.

Included in Section 1.6 of the CSA report are peer review comments on previous project reports and memoranda relating to both engineering geology and geotechnical studies for the project.

Section 1.6.1 includes the following engineering geology comments (italics) by CSA. Our responses and documentation of additional site investigation follow each item.

1. *Update seismic design parameters to the current codes (IBC 2009/CBC 2010).*

   The seismic design parameters were subsequently provided by GSI Soils in the "Updated Geotechnical Investigation", December 27, 2011

2. *Provide references and information regarding recent activity attributed to the Oceanic fault.*

   The Oceanic fault is the nearest fault to the site with documented evidence of recent seismic activity. The activity includes the December 2003, 6.5 magnitude San Simeon Earthquake attributed to the Oceanic fault by the U.S. Geological Survey. The San Simeon earthquake was caused by reverse faulting, and was centered approximately 22 miles north of the project site. (USGS Earthquake Hazards Program, 2003). A more recent 4.4 magnitude earthquake on June 20, 2009, occurred along the Oceanic fault according to the Geological Survey. No injuries were reported from the quake.

   The Oceanic fault is the northwestern segment of the West Huasna/Oceanic fault zone that trends north-northwest for approximately 100 kilometers extending from approximately the Santa Maria River on the south to San Simeon on the north. The Oceanic fault segment continues northward from near San Luis Obispo to San Simeon (County of San Luis Obispo

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3. *Provide a complete geologic map of the property illustrating the distribution of earth units at the ground surface, including limits of undocumented fill and other surficial deposits.*

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   CSA completed the longitudinal Cross Section 1–1' through existing exploration borings and test pits in the north half of the property and Cross Section 2-2' extending cross section B-B' previously completed by CHG (CSA, 2011). GSI Soils subsequently advanced Borings 4 and 5 near the southern property line and extended Cross Sections A-A' and B-B' (GSI, 2011). Cross Section B-B' (GSI, 2011) illustrates the shoring system and the location of engineered fill.

   Because no additional subsurface exploration was performed in the critical area near the driveway in the southern portion of the property, and near the westerly area of the proposed residential basement, CHG advanced four hand-auger borings and completed a longitudinal cross section near the southern property line (Figure 2, Cross Section D-D'). The existing Cross Section C-C' was updated (Figure 3) to include subsurface information obtained from boring HA-10. Cross Section B-B' has been updated herein (Figure 4) to include subsurface data from GSI Soils, Inc. (GSI Boring B-5) and to provide consistency with the new longitudinal Cross Section D-D'.

   The four borings completed by CHG along the southern property line were advanced using hand-auger equipment. Each boring was drilled until refusal occurred. Fill materials were observed to five feet depth in boring HA-9; to three feet depth in borings HA-10 and HA-10A; and to four and one half feet depth in boring HA-11. Fill materials consisted of silt, sand and gravel in boring HA-9, and mostly clay in the three remaining borings. Beneath the fill material, clayey gray to dark brown angular sandstone clasts were observed to the depth of refusal. These sandstones were interpreted to make up a thin weathered and broken veneer at the top of resistant

sandstone bedrock. The depth of refusal was interpreted to be the top of or very near the top of the sandstone bedrock. Lithologic logs for each of the four borings are attached.

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   A discussion of coastal hazards including shoreline erosion, wave runup, and coastal flooding northwest and immediately adjacent to the project site was provided by David W. Skelly of GeoSoils, Inc., March 14, 2011.

Please contact our office if you have any questions or comments.

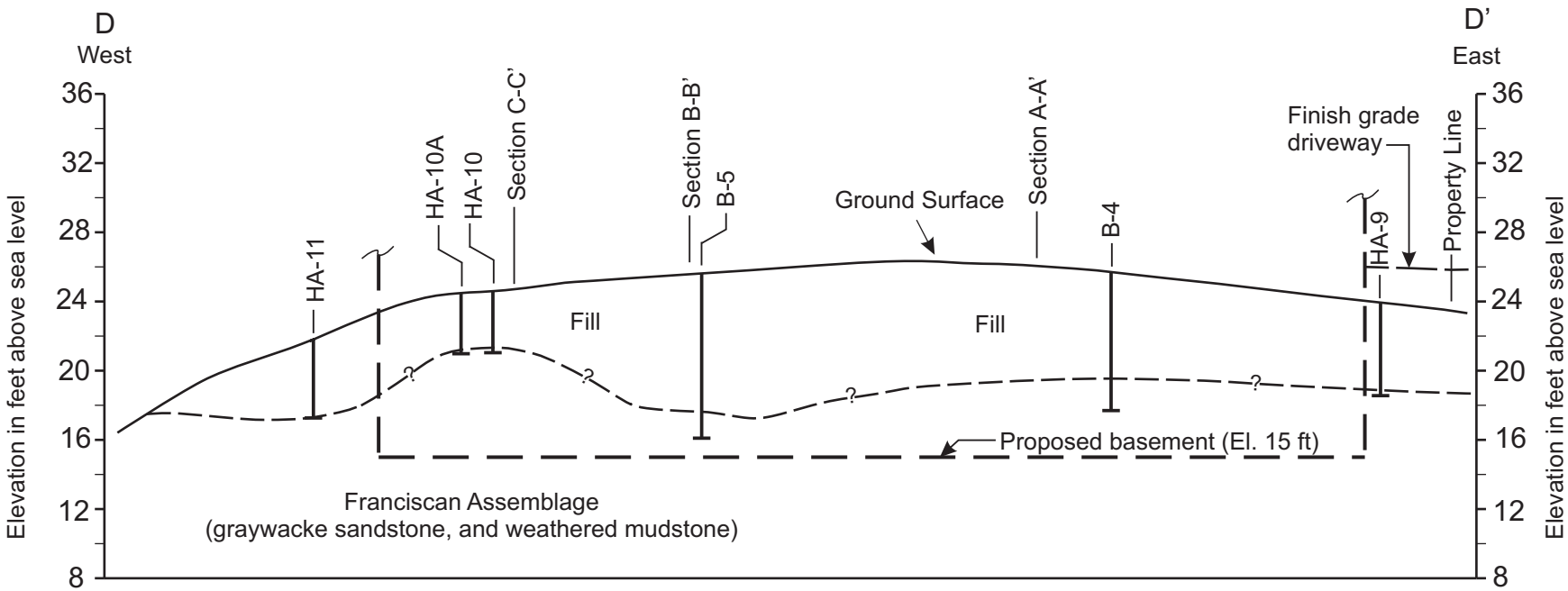
Sincerely,
CLEATH-HARRIS GEOLOGISTS, INC.

David R. Williams
Associate Geologist

Timothy S. Cleath
Certified Engineering Geologist #1102

Figure 1
Geologic Map
Loperena Property, Studio Drive
Cayucos, California
May 18, 2012

Cleath-Harris Geologists



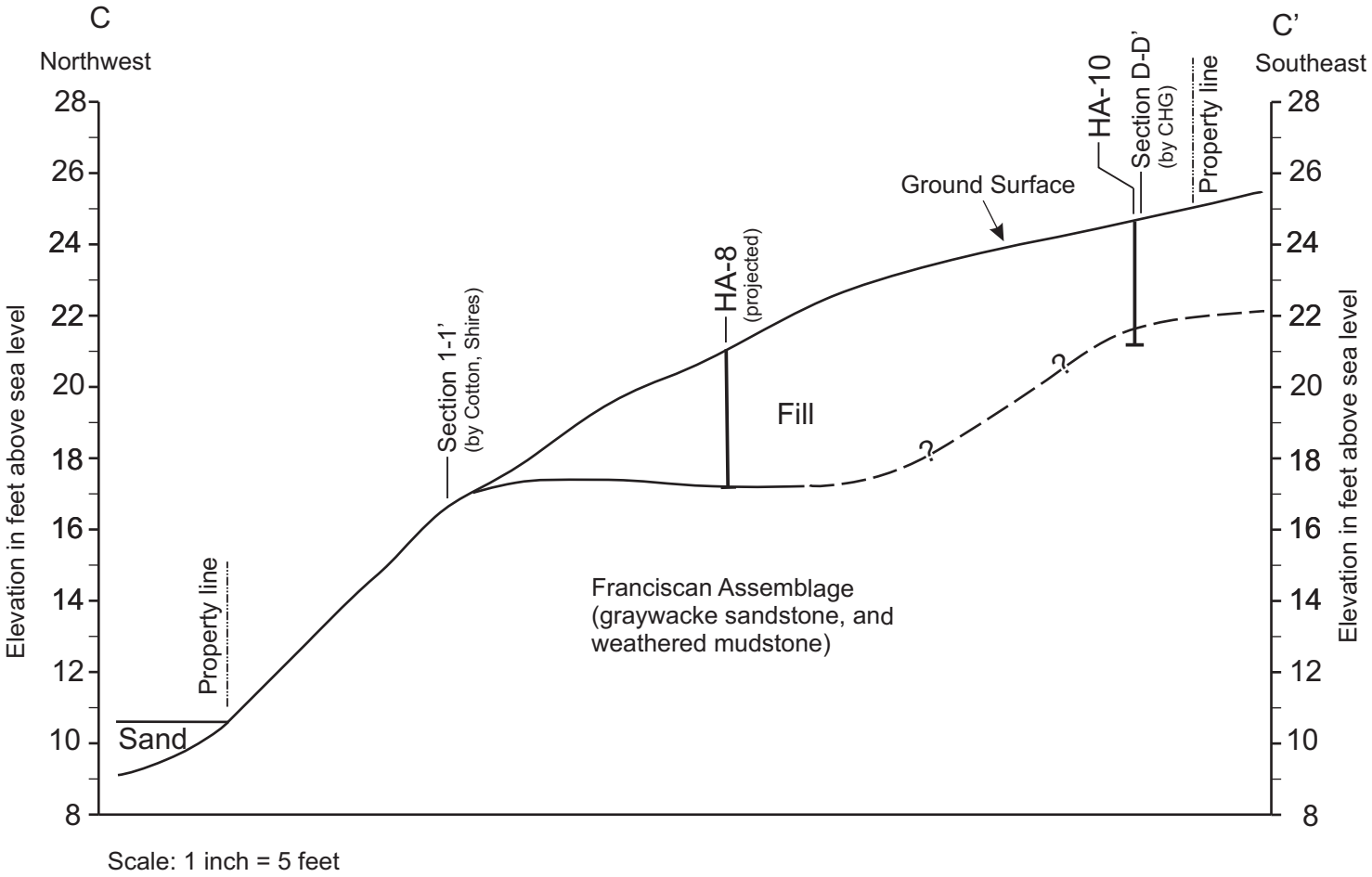
Scale: 1 inch = 10 feet

Explanation

HA-11	Hand auger boring location (by CHG)
B-5	Power auger boring location (by GSI)
-?	Geologic contact, queried and dashed where inferred

Figure 2
Cross Sections D-D'
Loperena Property, Studio Drive
Cayucos, California
May 18, 2012

Cleath-Harris Geologists



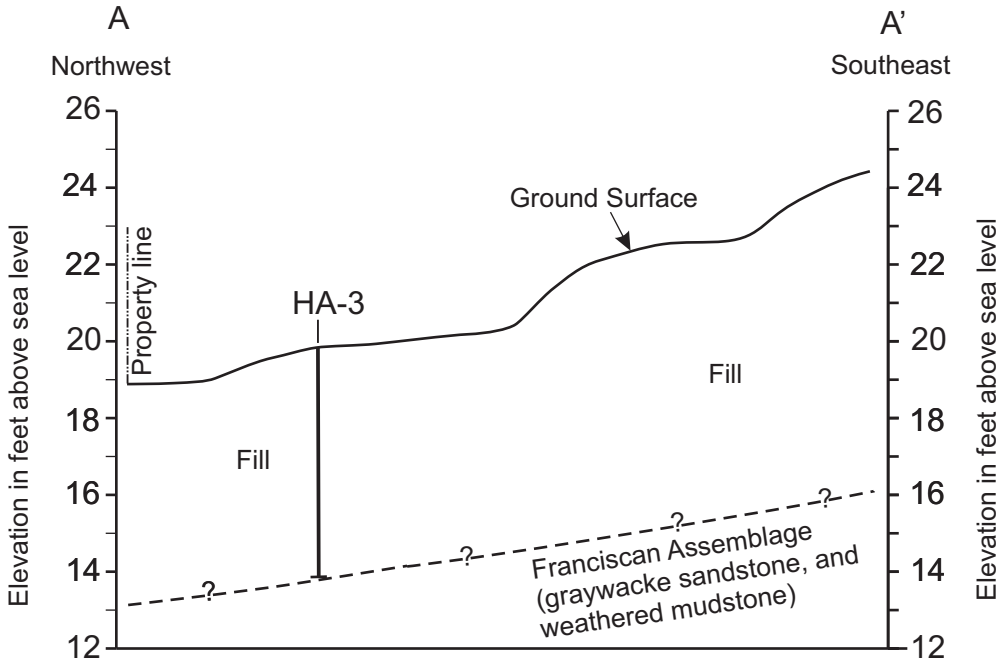
Explanation

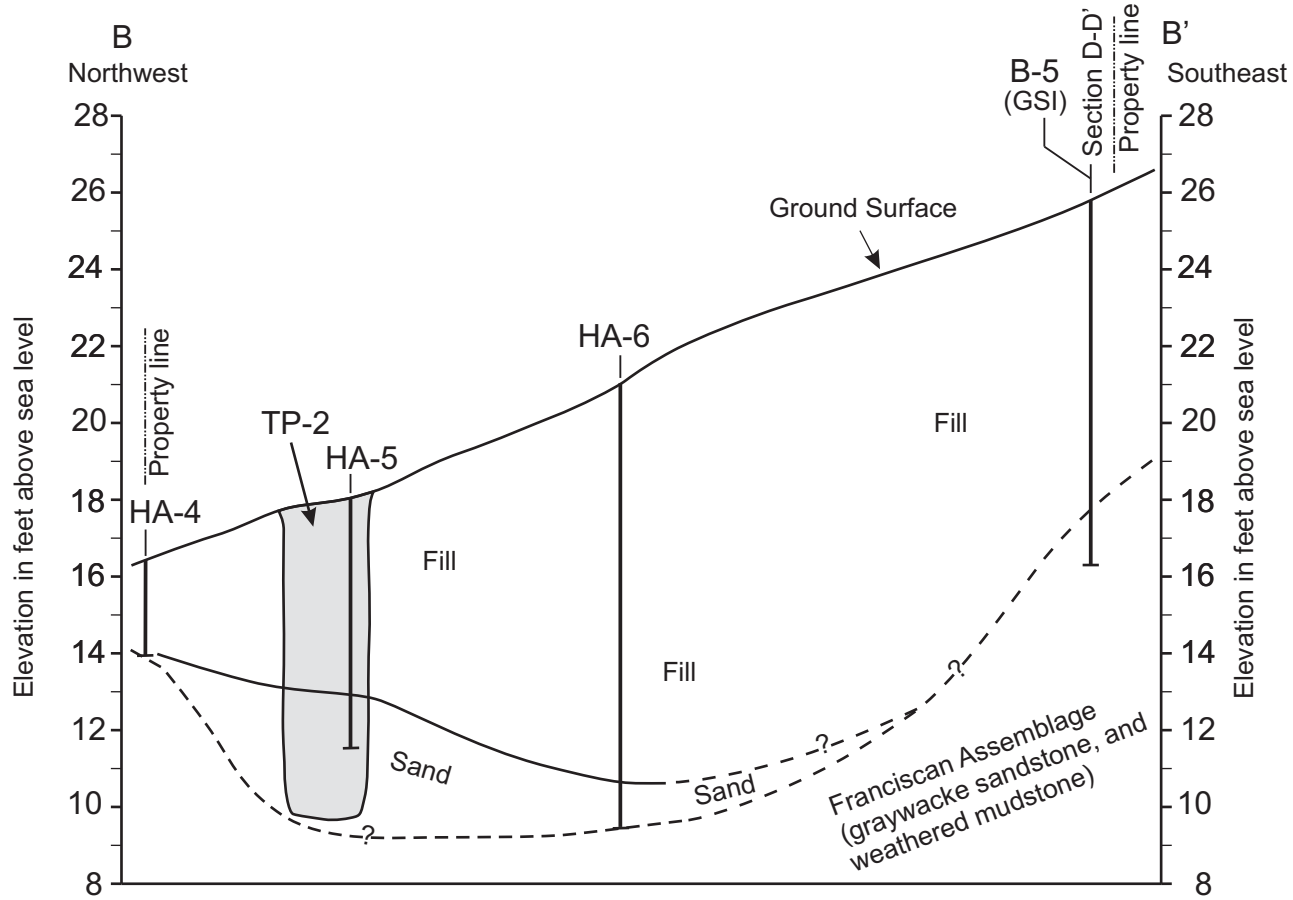
HA-10 Hand auger boring location

? Geologic contact, queried and dashed where inferred

Figure 3
Revised Cross Section C-C'
Loperena Property, Studio Drive
Cayucos, California
May 18, 2012

Cleath-Harris Geologists





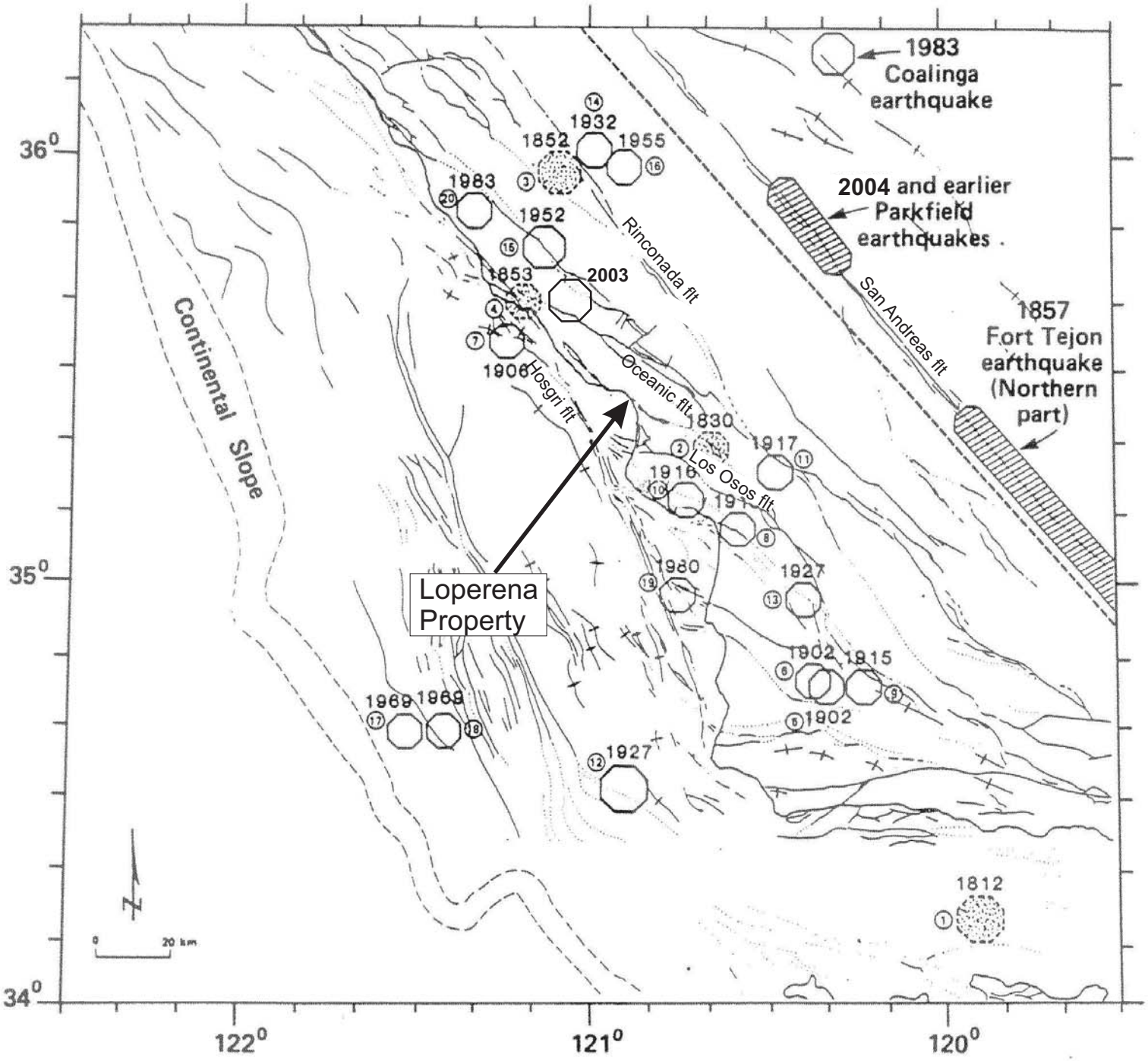
Explanation

HA-6	Boring designation
TP-2	Backhoe test pit designation
?	Geologic contact, queried and dashed where inferred

Scale: 1 inch = 5 feet

Figure 4
Revised Cross Sections A-A' and B-B'
Loperena Property, Studio Drive
Cayucos, California
September 19, 2012

Cleath & Associates



EXPLANATION

Earthquake epicenter:

1812 – Year of event

① Event number

Event location that is poorly constrained

Event location that is within 20 kilometers and generally within 10 kilometers

Magnitude

5.0 – 5.9

6.0 – 6.9

7.0 – 7.9

Base Map: Diablo Canyon Power Plant
Long Term Seismic Program - Final Report
Pacific Gas & Electric, July 1988

0 40 80

Scale: 1 inch = 40 kilometers

Figure 5

Earthquake Epicenter Map
Loperena Property

Cleath-Harris Geologists

Attachments

Lithology Logs
of 2012 Hand–Auger Borings

Boring Logs
Loperena Property, Studio Drive, Cayucos, California

Date of borings: May 10, 2012
Geologists: Williams, Pfeifle; Cleath-Harris Geologists.
Drilling Method: hand auger

Boring HA-9
Elevation: 23.7 ft above sea level (from topographic map)
Total depth: 5.5 feet
Depth to top and bottom in feet

Top	Bottom	Thickness	Description
0	1.5	1.5	**FILL;** silty sand with gravel; dark brown; fine grained sand; moist.
1.5	3	1.5	**FILL;** sand with silt and gravel; dark brown; fine to medium grained sand; gravel to 3”, subrounded; fine roots; moist.
3	5	1	**FILL;** gravely sand with silt; dark brown; fine to medium grained sand; gravel to 3”, angular to rounded; moist.
5	5.5	0.5	**Sandstone;** greywacke sandstone fragments; gray to dark brown; moist.

Refusal at 5.5 feet depth.

Boring Logs
Loperena Property, Studio Drive, Cayucos, California

Date of borings: May 10, 2012
Geologists: Williams, Pfeifle; Cleath-Harris Geologists.
Drilling Method: hand auger

Boring HA-10
Elevation: 24.2 ft above sea level (from topographic map)
Total depth: 3.5 feet
Depth to top and bottom in feet

Top	Bottom	Thickness	Description
0	1	1	**FILL;** sandy clay; dark brown, variegated dark red brown; soft; fine to medium sand; moist.
1	2	1	**FILL;** sandy clay with gravel; dark brown; soft; fine to medium sand; gravel to 2”, roots and galvanized wire; moist.
2	3	1	**FILL;** clay with sand and gravel; stiff; fine sand; greywacke gravel to 1”, angular; moist.
3	3.5	0.5	**Sandstone;** greywacke sandstone fragments; gray to dark brown; moist.

Refusal at 3.5 feet depth.

Boring Logs
Loperena Property, Studio Drive, Cayucos, California

Date of borings: May 10, 2012
Geologists: Williams, Pfeifle, Cleath-Harris Geologists.
Drilling Method: hand auger

Boring HA-10A
Elevation: 24.2 ft above sea level (from topographic map)
Total depth: 3.5 feet
Depth to top and bottom in feet

Top	Bottom	Thickness	Description
0	1	1	**FILL;** sandy clay; dark brown, variegated dark red brown; soft; fine to medium sand; moist.
1	2	1	**FILL;** sandy clay with gravel; dark brown; soft; fine to medium sand; gravel to 2”, roots and galvanized wire; moist.
2	3	1.5	**FILL;** clay with sand and gravel; stiff; fine sand; greywacke gravel to 1”, angular; moist.
3	3.5	0.5	**Sandstone:** greywacke sandstone fragments; gray to dark brown; moist.

Refusal at 3.5 feet depth.

Boring Logs
Loperena Property, Studio Drive, Cayucos, California

Date of borings: May 10, 2012
Geologists: Williams, Pfeifle, Cleath-Harris Geologists.
Drilling Method: hand auger

Boring HA-11
Elevation: 21.8 ft above sea level (from topographic map)
Total depth: 4.5 feet
Depth to top and bottom in feet

Top	Bottom	Thickness	Description
0	2	2	**FILL;** clay with sand; variegated redish brown to dark brown; soft; fine sand; moist.
2	3	1	**FILL;** clayey sand; dark brown; medium consistency; fine sand; roots; moist.
3	3.5	0.5	**FILL;** sandy clay with rock fragments; dark brown to black; soft consistency; angular; moist.
3.5	4	0.5	**FILL;** clay; yellowish brown; stiff consistency; moist.
4	4.5	0.5	**FILL;** sandy clay with gravel; yellowish brown; fine to medium grained sand; greywacke sandstone fragments to 1”; refusal on flat surface at 4.5’.

Refusal at 4.5 feet depth.

SHORELINE ENGINEERING
STRUCTURAL AND CIVIL

20 September 2012
#293-02

Shawna Scott
SWCA Environmental Consultants
1422 Monterey Street, Suite C200
San Luis Obispo, CA 934010

re: Loperena, County of San Luis Obispo
Responses to Supplemental Geotechnical Peer Review For EIR Preparation, 8/21/12

Engineering Geology/Geotechnical Engineering Comments
Refer to the attached report prepared by Cleath Harris Geologists, September 19, 2012.

Civil/Structural Plan Comments

1. *Note 4 on the Shoring Detail (sheets 4 & 5) should reference the new subsurface data....*
   Sheets 4 & 5 have been revised to include information from the new subsurface data. While it appears the sandstone bedrock layer may be shallower, the soldier beam piles were not revised to reflect the possibility of variations in sandstone bedrock depth at the time drilling is performed.
2. *The plans by Shoreline Engineering are unclear as to whether the driveway is to be supported on engineered fill.....*
   The question as to whether or not the driveway would be a structural slab or supported by fill was addressed in correspondence, 5 April 2011, Item 4. The following text is taken from the correspondence:
   4. *Foundation excavation.*
      a. *The foundation cut is approximately 400 cy.*
      b. *The fill in the driveway is approximately 150 cy.*
      c. *Net excess to be transported off the site is approximately 250 cy.*

   It was determined then that the driveway would be bounded on three sides by retaining walls, and the driveway to be placed over fill.

Be in touch with any questions or comments.

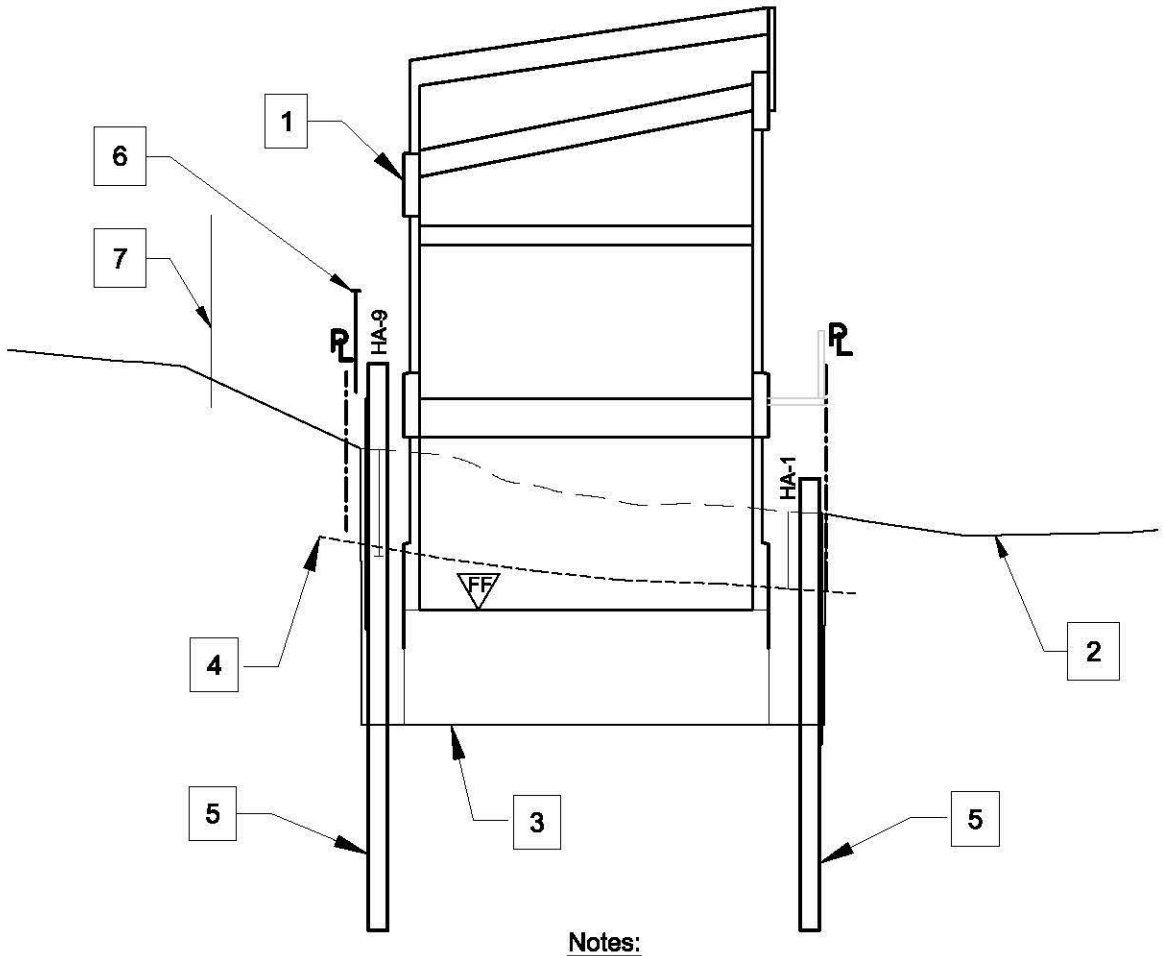
Respectfully submitted,

Bruce Elster, PE

Attachment: Update #2 to Engineering Geology Reports for the Proposed Loperena Residence, Lot 41, Studio Drive, Cayucos, CA. Cleath-Harris Geologists, Inc..
Revised Shoring Detail Sheets 4 & 5

copy: C Novak

505 HARBOR STREET, MORRO BAY, CA 93442 P. 805 772 6466 F. 772.6467



Notes:

1. Proposed residence, typ.
2. EG @ SL-1.
3. Concrete deadman, 6' deep.
4. Graywacke sandstone level per Cleath & Harris 1/12/2007 & 9/19/2012.
5. Soldier beam shoring system w/ W12x53 @ 8' and steel traffic plate lagging. Layout per plan.
6. Safety fencing.
7. Edge of neighbooring residence (e).

Explanation:
HA-xx: Boring by CHG

4

Shoring Detail @ SL-1 (Looking westerly)

Scale: 1"=10'

Shoreline Engineering
Structural & Civil
505 Harbor Street
Morro Bay, CA 93442
(805) 772-6466 v
(805) 772-6467 f
be@shoreline-engineering.net

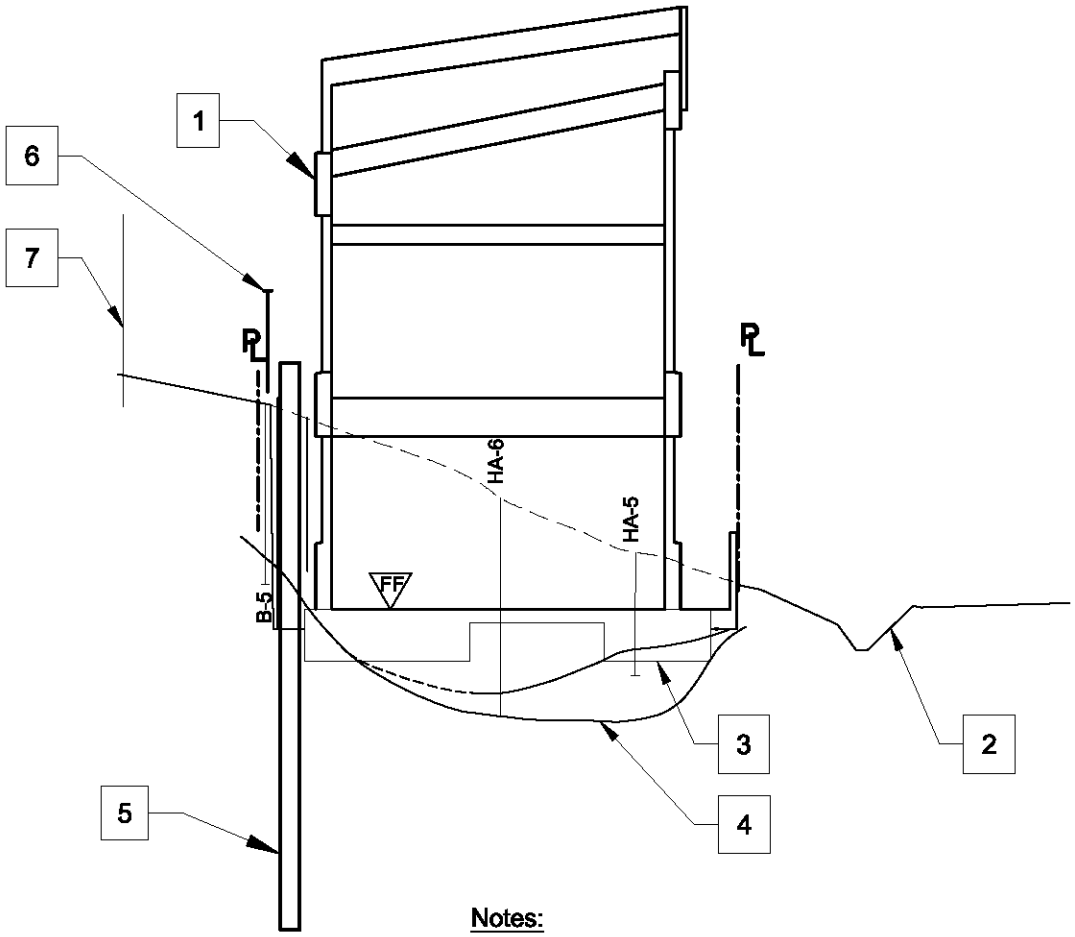
Shoring Detail

Loperena Residence
Studio Drive, Cayucos, CA

Job No.:	Dwg No.:	Scale:
293-02	—	1"=10'

Sht 4 of 5

Date: Jan 2012



Notes:

1. Proposed residence, typ.
2. EG @ SL-2.
3. Approx. level of construction pad.
4. Graywacke sandstone level per Cleath & Harris 9/19/2012.
5. Soldier beam shoring system w/ W12x53 @ 8' and steel traffic plate lagging. Layout per plan.
6. Safety fencing.
7. Edge of neighbouring residence (e).

Explanation:

B-xx: Boring by GSI
HA-xx: Boring by CHG
Borings are between 4 ft & 5 ft off axis easterly of SL-2 in the longitudinal direction, and are considered representational of condition below SL-2.

5

Shoring Detail @ SL-2 (Looking westerly)

Scale: 1"=10'

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Shoring Detail

Loperena Residence
Studio Drive, Cayucos, CA

Job No.:	Dwg No.:	Scale:
293-02	—	1"=10'

Sht 5 of 5

Date: Jan 2012

SHORELINE ENGINEERING
STRUCTURAL AND CIVIL

Engineering Evaluation
Studio Drive Residence
Cayucos
APN 064-253-007
County of San Luis Obispo, CA

REGISTERED PROFESSIONAL ENGINEER
BRUCE SURTEES ELSTER
No. 32981
CIVIL
STATE OF CALIFORNIA

For:
Jack Loperena
2764 West Athens
Fresno, CA 93711

January 2012
#293-02

Shoreline Engineering, Inc
505 Harbor Street
Morro Bay, CA 93442
805-772-6466
be@shoreline-engineering.net

Table of Contents

Table of Contents 2
Project Description 3
Design Parameters 3
Structural Evaluation Summary 4
Limitations & General Conditions 4
Model Sketches 5
Wind Generation Parameters 9
Seismic Generation Parameters 10
Basic Load Cases 11
Load Combinations 12
Code Check, Strength, Vertical Load Combinations, Full Uniform Loading 16
Code Check, Strength, Vertical w/ Wind Load Combinations, Skip Uniform Loading 19
Code Check, Vertical w/ Seismic Load Combinations, Skip Uniform Loading 23
Model Sketch w/ Nodes indicated for use in Service Check 27
Deflection Check, Unfactored Service Loads 28
Foundation Model Sketch 29
Load Combinations, Foundation 30
Foundation Soil Pressure Diagrams 32
Soldier Beam Check 36

Total Sheets 38

Project Description

The project is generally described as the construction of a single-family residence on a 25 foot wide lot located at the northerly end of Studio Drive in the community of Cayucos. It is listed on the Assessor's Roles as APN 064-253-007. The evaluation has been prepared for the purpose of demonstrating structural engineering feasibility of the residential structure submitted for review and approval by the County of San Luis Obispo. It is anticipated that modifications will be made to economize the structural system at the time final engineering plans are prepared for building permit purposes.

The structure of the residence includes:

1. A basement level. Exterior walls are of concrete, and retain soils along the southerly, easterly, and northerly sides of the residence.
2. A main living area level with a mezzanine above the kitchen area.
3. The superstructure of the residence is primarily steel framed.
4. The foundation system consists of:
   a. slab-on-grade continuous footings supporting the south, west, and north walls.
   b. deadman foundation along the east frontage wall. The deadman is approximately 18 ft x 18 ft x 6 ft deep. The purpose of the deadman is to resist overturning forces generated primarily by the cantilevered portion of the residence.
5. Temporary excavation support is to be provided by steel soldier beams installed in drilled holes filled with lean concrete. The soldier beams are to be lagged with steel plates to provide support to adjacent properties during construction activities. The soldier beams and lagging will be removed at the completion of backfill operations.

Design Parameters

The following parameters have been utilized in the preparation of this engineering analysis:

1. Code: California Building Code, Title 24. (CBC). 2010 Edition.
   a. Wind Loads. CBC §1609.
   b. Seismic Loads: Equivalent Static Force Method.
   c. ASCE 7-05: Minimum Design Loads per Vol 2.1, pg ix.
   d. ACI 318-08. Building Code Requirements for Structural Concrete & Commentary. American Concrete Institute.
   e. AISC 341-05. Seismic Design Manual. American Institute of Steel Construction.
   f. AISC 360-05: Manual of Steel Construction. American Institute of Steel Construction.
   g. NDS 05. National Design Specification, 2005. American Forest & Paper Association. Utilized for wood design criteria.
   h. Reference codes are incorporated into the California Building Code by reference.
2. Site topographic survey provided by Volbrecht Surveys.
3. A geotechnical investigation prepared by GSI Soils, Inc. (Project 6-4210, dated December 19, 2011), was utilized by the engineer for the purpose of preparing the engineering evaluation. The geotechnical investigation is incorporated into the evaluation by this reference. For the purpose of preparing the engineering report, this engineer has used the following information:
   a. Near surface soils consist of undocumented fill materials containing debris to a depth of 4.5 to 6 feet in depth. These soils are to be removed from the site.
   b. The foundation is to be keyed into the graywacke sandstone layer with a minimum penetration into the sandstone of 12 inches.
   c. Excavation and replacement of soils under the foundation area of the structure and related site concrete work shall be performed in accordance with the Geotechnical Investigation, and supervised by an engineer.
   d. Potential for Liquifaction: Negligible
   e. Foundation Soil/Rock Description: Graywacke Sandstone
   f. Short Period Acceleration coefficient, S_{DS} : 0.98g
   g. One-Second Acceleration, S_{D1}: 0.491g
   h. Site Class: C
   i. Modulus of Sub-grade Reaction, k: 150 pci
   j. Soil bearing pressure allowable (D+L), SP: 3,500 psf (may be increased per report
   k. Lateral bearing pressure, P_p: 400 pcf
   l. Lateral bearing pressure, P_p, Graywacke: 600 pcf (temp const support)
   m. Lateral Pressure, level-granular backfill, P_a: 30 pcf
   n. At-Rest Lateral Pressure, P_{ar}: 50 pcf
   o. Lateral sliding resistance: 0.35
   p. Soils sulfate content is in the negligible range.
4. Additional soil parameters utilized by the engineer include:
   a. Soil expansive index, EI: 0 to <10 (select fills)
   b. When friction & passive pressures are combined, the lesser value shall be reduced by 33%.
   c. Allowable soil pressures (DL + LL) may be increased by 1/3 for short-term wind and seismic loads.
5. Loads:
   a. Dead Load

Structure, as modeled:	Internal
b. Live Load	
Floor & Deck	50 psf
Roof	20 psf (unreduced)
c. Wind Speed:	85 mph
d. Seismic: Refer to Calc Sheet	

6. Load Combinations:
   a. ASCE 7. Refer to Calc Sheets
   b. Live loads were skip loaded to check for variability of structural response.
7. In the event any information contained in this analysis is not in keeping with actual field conditions, the engineer shall be notified immediately for direction.

Structural Evaluation Summary

1. 20 basic load cases were included in the evaluation.
2. 70 load combinations were evaluated for system strength.
3. 8 load combinations were evaluated for serviceability, or system deflection. For the purpose of evaluating system feasibility, this evaluation was limited to checking the cantilevered portion of the structure.
4. 26 load combinations were evaluated for foundation stability and soil bearing pressure.
5. A total of 104 load combinations were evaluated in both North-South and East-West directions, and checked for eccentric loading conditions.
6. The engineering evaluation includes the results from those combinations that produce controlling results. Inclusion of all results data would produce an unmanageable amount of data. In the event there is interest to review additional material, it may be made available upon written request.
7. Examination of results indicates the structural system, as proposed, can provide acceptable support and service for the proposed residence.

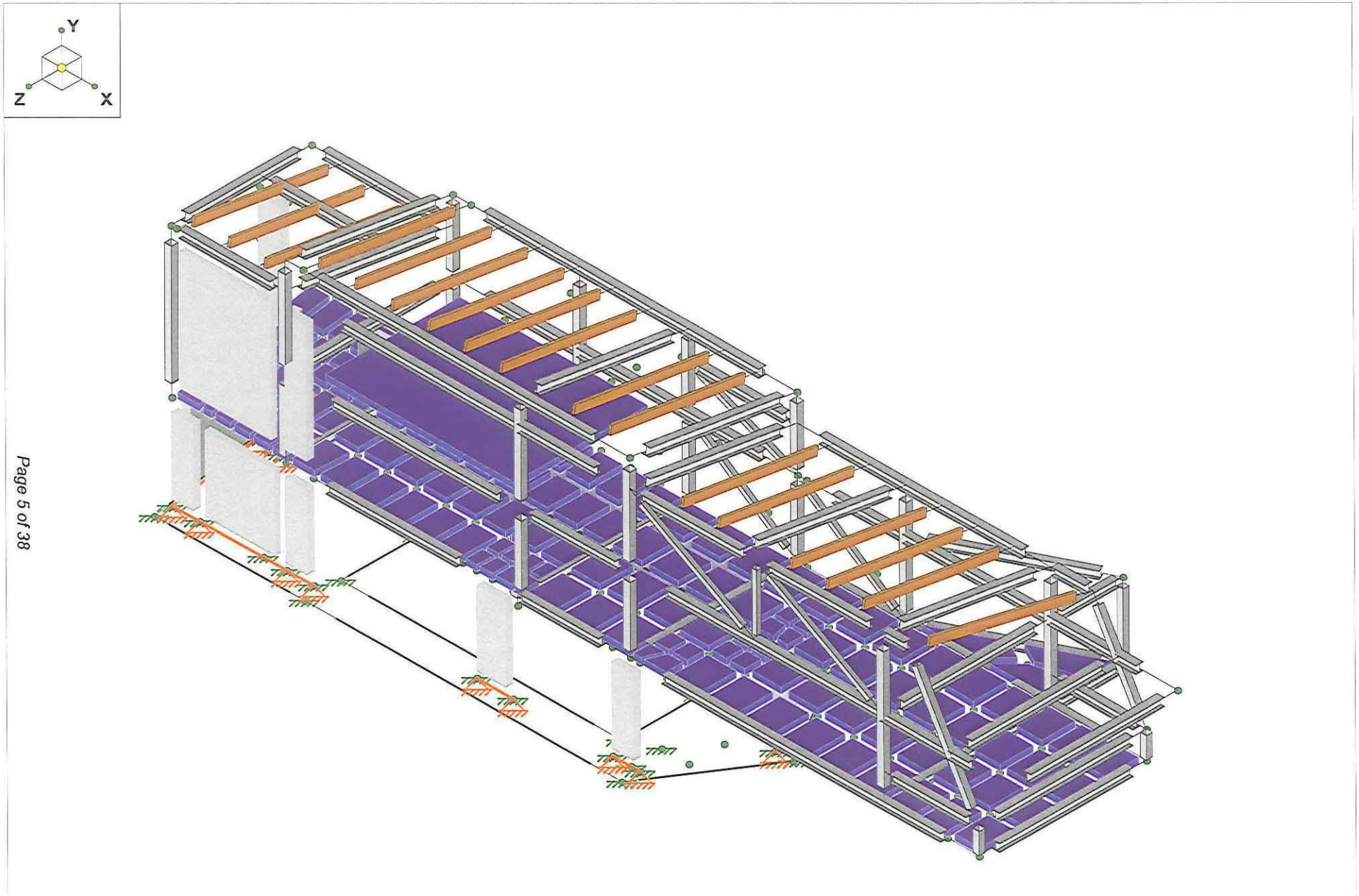
Limitations & General Conditions

These evaluations, calculations, and drawings are the instruments of service of the engineer, and are the property of Shoreline Engineering, Inc. The information contained herein is solely for use in this specific project and site, and shall not be used otherwise without the express written permission of Shoreline Engineering, Inc. Evaluations, drawings, and the information contained thereon, together with the represented ideas shall remain the property of the engineer, and in no way shall be copied or used in connection with any other project without the express written consent of the engineer. Visual contact with this notice shall constitute prima facie evidence of acceptance of these restrictions.

Information contained in this evaluation is limited solely to that information relating to the structure; and does not include any items not specifically included in the project documents prepared by the engineer.

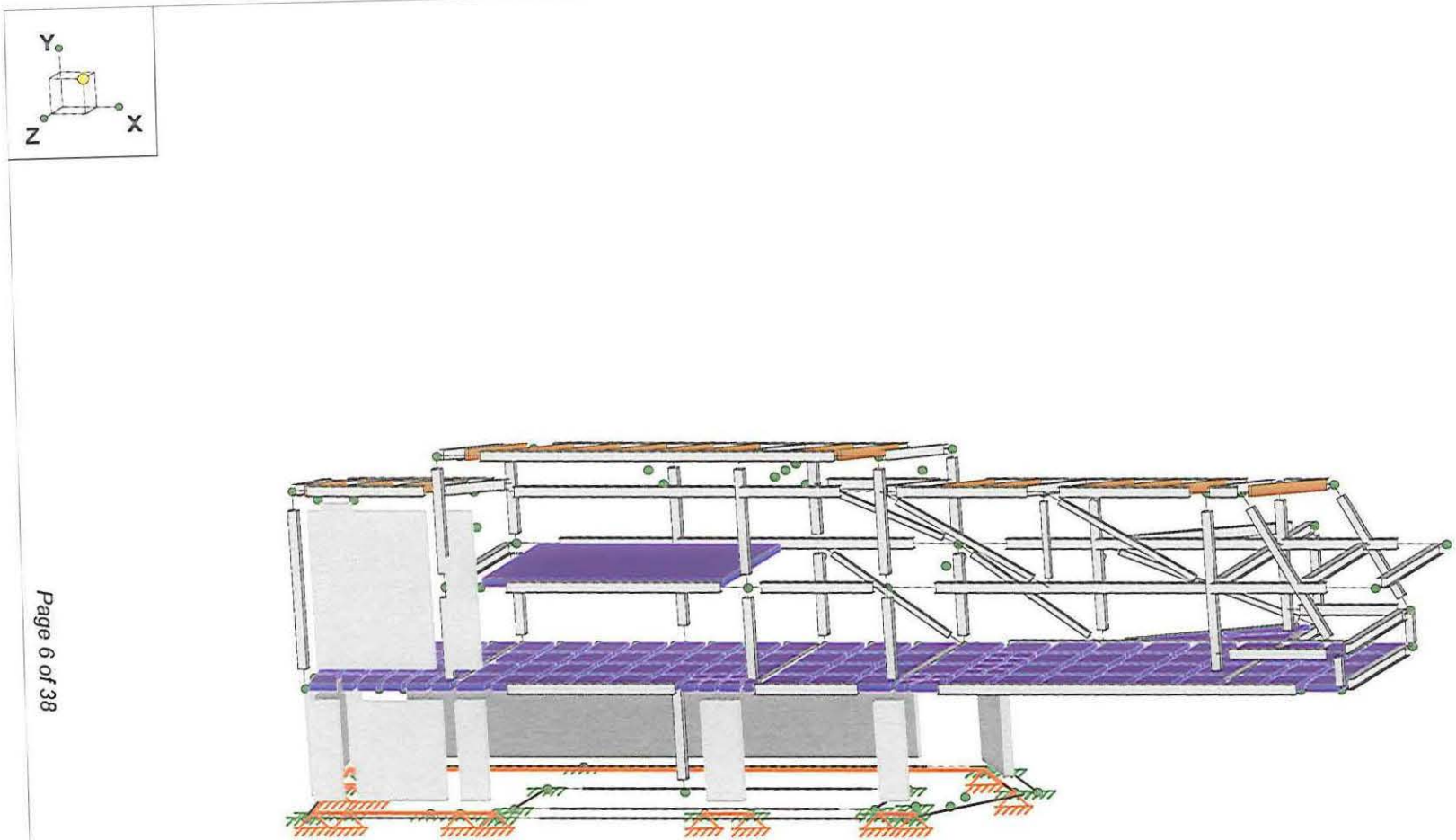
It is the goal of the lateral seismic analysis contained herein to provide the structure with sufficient strength to resist a minor earthquake without significant damage, and a major earthquake without collapse. An "earthquake proof" structure is not feasible, nor required by the Code.

Structural drawings prepared by the engineer are preliminary and illustrative of the proposed system to be used in constructing the residence. The structural drawings are not intended to be used in construction.



WHOLE STRUCTURE, ISOMETRIC

Solution: Envelope



WHOLE STRUCTURE, LOOKING SOUTHERLY

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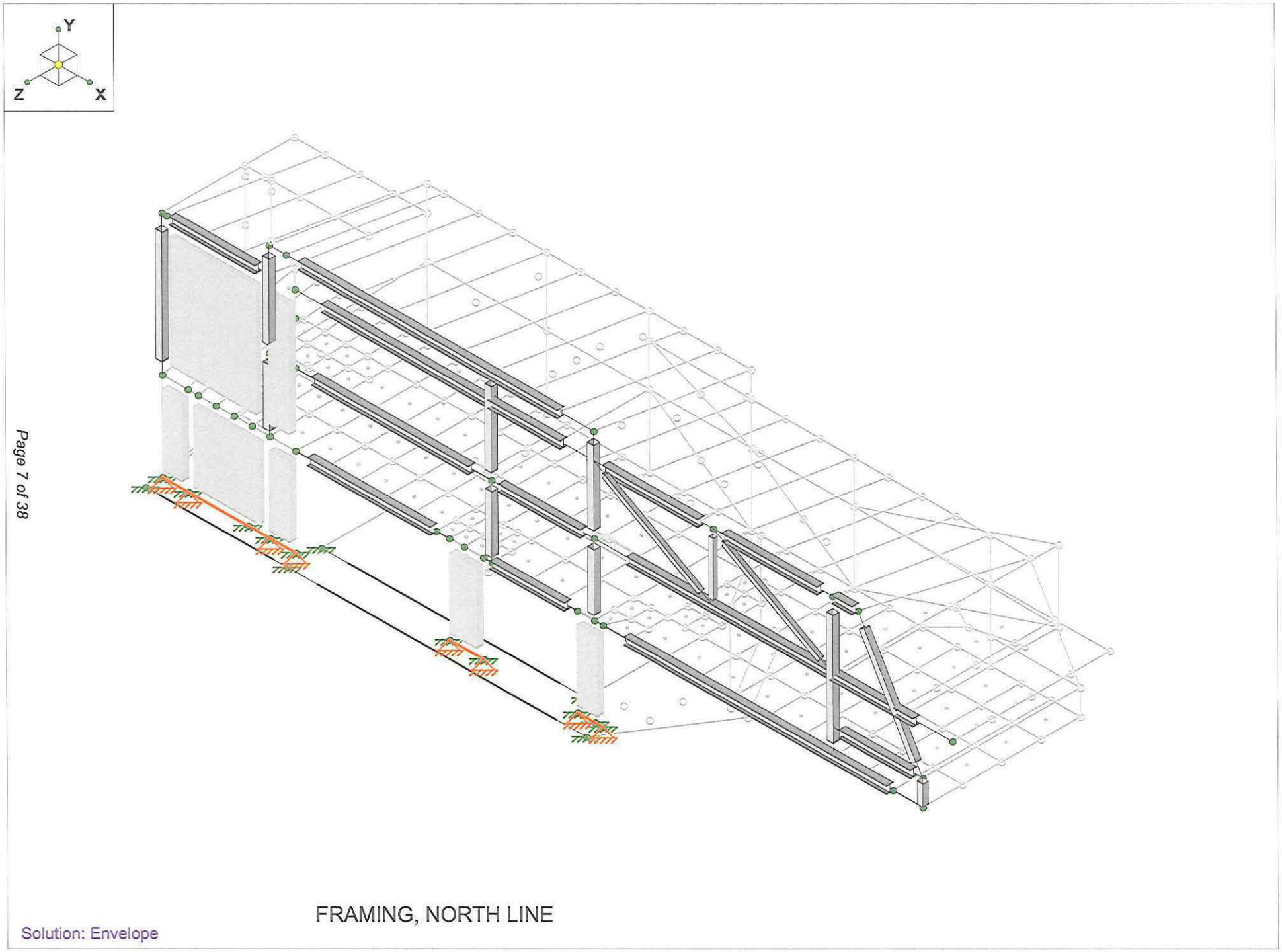
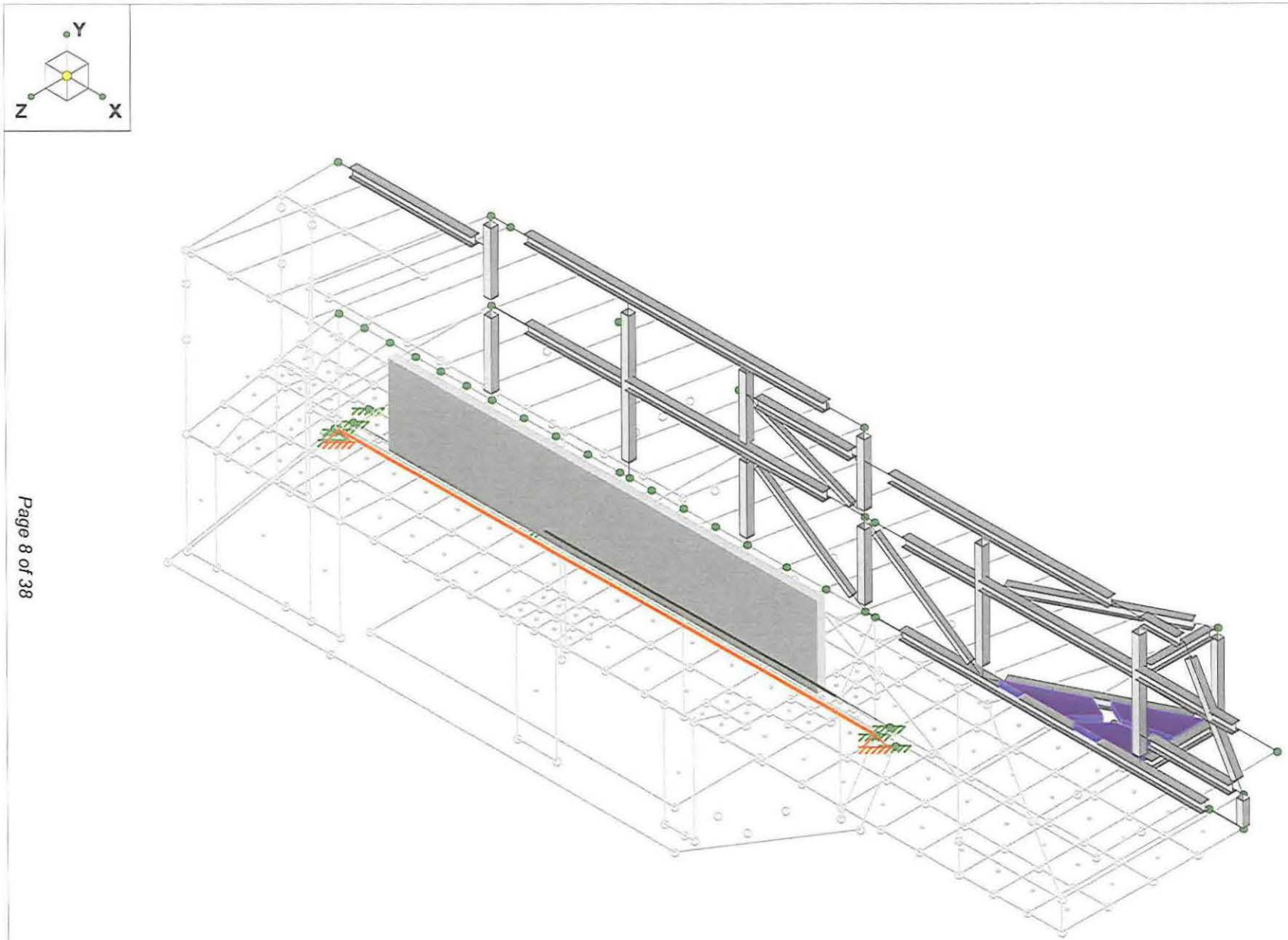


FRAMING, NORTH LINE

Solution: Envelope



FRAMING, SOUTH LINE

Solution: Envelope

Wind Generation Input

Wind Code:	ASCE 7-05	Topographic Factor K1:	.17
Wind Speed, V(mph):	90	Topographic Factor K2:	1
Occupancy Category:	I	Topographic Factor K3:	1
Exposure Category:	D	Directionality Factor Kd:	.85

Wind Generation Detail Results

Importance Factor:	.87	Kzt:	1.369
Exposure Constant Alpha:	11.5	h (ft):	30
Exposure Constant zg:	700	Kh:	1.162
Gust Effect Factor, G:	.85	Windward Cp:	.8
		qh (ksf):	.024

Wind Generation Floor Geometry Results

Floor Level	Height (ft)	Kz	Width (Z) (ft)	Length (X) (ft)	Leeward Cp(Z)	Leeward Cp(X)
Diaphragm : 1	30	1.162	18.58	38	.5	.298
Diaphragm : 2	27	1.141	18.58	86.25	.5	.2
Diaphragm : 3	19	1.073	26.58	94.464	.5	.222
Diaphragm : 4	10.5	1.03	26.58	92.08	.5	.227

Wind Generation Floor Force Results

Floor Level	qz (ksf)	Windward Pres. (ksf)	Leeward Pres. Z (ksf)	Leeward Pres. X (ksf)	Force Z (k)	Force X (k)
Diaphragm : 1	.024	.017	.01	.006	1.537	.634
Diaphragm : 2	.024	.016	.01	.004	12.645	2.088
Diaphragm : 3	.023	.015	.01	.005	20.022	4.371
Diaphragm : 4	.022	.015	.01	.005	21.934	4.901

Seismic Generation Input

Seismic Code:	**ASCE 7-05**				
Ct_Z:	**.028**	T_Z (sec):	**.2**	R_Z:	**3.5**
Ct_X:	**.028**	T_X (sec):	**.2**	R_X:	**3.5**
Ct Exp. Z:	**.8**	Ct Exp. X:	**.8**	Seismic Weight LC:	**23 Seis DL**
Occupancy Cat:	**I or II**	TL (sec):	**8**		
SD1 (g):	**.302**	SDS (g):	**.761**	S1 (g):	**.454**
Base Elev (ft):	**0**				

Seismic Generation Detail Results

T_Z Used (sec):	**.2**	T_Z Method A:	**.425**	T_Z Upper Limit:	**.596**
T_X Used (sec):	**.2**	T_X Method A:	**.425**	T_X Upper Limit:	**.596**
Importance Fac.:	**1**	Design Cat.:	**D**		
V_Z (k):	**161.862**	Gov. Eqn.	**ASCE Eqn 12.8-2**		
V_X (k):	**161.862**	Gov. Eqn.	**ASCE Eqn 12.8-2**		

Seismic Generation Force Results

Floor Level	Height (ft)	Weight (k)	Force Z (k)	Force X (k)	CG Z (ft)	CG X (ft)
Diaphragm : 1	**30**	**72.306**	**29.197**	**29.197**	**14.898**	**35.045**
Diaphragm : 2	**27**	**143.436**	**52.128**	**52.128**	**12.191**	**44.975**
Diaphragm : 3	**19**	**136.439**	**34.893**	**34.893**	**12.227**	**39.642**
Diaphragm : 4	**10.5**	**322.967**	**45.645**	**45.645**	**11.477**	**41.86**
Base		**69.293**				

Seismic Generation Diaphragm Results

Floor Level	Width (Z) (ft)	Length (X) (ft)	Z Plus (ft)	Z Minus (ft)	X Plus (ft)	X Minus (ft)
Diaphragm : 1	**18.58**	**38**	**.929**	**.929**	**1.9**	**1.9**
Diaphragm : 2	**18.58**	**86.25**	**.929**	**.929**	**4.313**	**4.313**
Diaphragm : 3	**26.58**	**94.464**	**1.329**	**1.329**	**4.723**	**4.723**
Diaphragm : 4	**26.58**	**92.08**	**1.329**	**1.329**	**4.604**	**4.604**

BASIC LOAD CASES

	Description	Category	X	Y	Z	Joint	Point	Distributed	Area (Member)	Surface (Plate/Wall)
1	Dead Load	DL		-1				17	3	
2	Basem't Lev LL	LL								
3	Main Lev LL	LL								134
4	Mezzanine Lev LL	LL								1
5	Roof LL	RLL							3	
6	Main Lev LL Skip	OL1								52
7	Roof LL Skip	OL2							1	
8	Soil Load	EPL								3
9	Wind Load Z	WLZ				4				
10	Wind Load X	WLX				4				
11	Partial Z Wind Load 1	WLZP1				4				
12	Partial Z Wind Load 2	WLZP2				4				
13	Partial X Wind Load 1	WLXP1				4				
14	Partial X Wind Load 2	WLXP2				4				
15	Earthquake Load Z	ELZ				4				
16	Earthquake Load X	ELX				4				
17	Earthquake Load Z Plus X Eccentr	ELZ+X				4				
18	Earthquake Load Z Minus X Eccent	ELZ-X				4				
19	Earthquake Load X Plus Z Eccentr	ELX+Z				4				
20	Earthquake Load X Minus Z Eccent	ELX-Z				4				

Notes on Load Combinations:
1. X, Y, Z: Factors applying default mass properties in the indicated global axis direction.
2. Joint, Point, Distributed, Area, Surface: Table indicates the number of application instances in the model.

LOAD COMBINATIONS
per ASCE 7-05. Refer to Notes for comments

	Description	Solve	P-Delta	SRSS	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
	Unif LL													
2	ASCE 1		Y		DL	1.4								
3	ASCE 2 (a)		Y		DL	1.2	LL	1.6	RLL	0.5	EPL	1.6		
4	ASCE 2 (b)		Y		DL	1.2	LL	1.6	EPL	1.6				
5	ASCE 3 (a)		Y		DL	1.2	RLL	1.6	LL	1				
	Wind Comb													
8	ASCE 3 (b) (a)		Y		DL	1.2	RLL	1.6	WLX	0.8	WLX+R	0.8		
9	ASCE 3 (b) (b)		Y		DL	1.2	RLL	1.6	WLZ	0.8	WLZ+R	0.8		
10	ASCE 3 (d) (a)		Y		DL	1.2	WLX	0.8	WLX+R	0.8				
11	ASCE 3 (d) (b)		Y		DL	1.2	WLZ	0.8	WLZ+R	0.8				
12	ASCE 3 (f) (a)		Y		DL	1.2	RL	1.6	WLZ	0.8	WLZ+R	0.8		
13	ASCE 4 (a) (a)		Y		DL	1.2	WLX	1.6	LL	1	RLL	0.5	WLX+R	1.6
14	ASCE 4 (a) (b)		Y		DL	1.2	WLZ	1.6	LL	1	RLL	0.5	WLZ+R	1.6
15	ASCE 4 (b) (a)		Y		DL	1.2	WLX	1.6	LL	1	WLX+R	1.6		
16	ASCE 4 (b) (b)		Y		DL	1.2	WLZ	1.6	LL	1	WLZ+R	1.6		
17	ASCE 4 (c) (a)		Y		DL	1.2	WLX	1.6	LL	1	WLX+R	1.6		
18	ASCE 4 (c) (b)		Y		DL	1.2	WLZ	1.6	LL	1	WLZ+R	1.6		
19	ASCE 6 (a)		Y		DL	0.9	WLX	1.6	WLX+R	1.6				
20	ASCE 6 (b)		Y		DL	0.9	WLZ	1.6	WLZ+R	1.6				

LOAD COMBINATIONS
per ASCE 7-05. Refer to Notes for comments

	Description	Solve	P-Delta	SRSS	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
	Eqk Comb													
23	Seis DL	1	Y		DL	1		LC 23 used for purpose of setting up seismic mass						
24	ASCE 5 (a)	1	Y		DL	1.2	Sds*DL	0.2	Rho*ELX	1	LL	1		
25	ASCE 5 (b)	1	Y		DL	1.2	Sds*DL	0.2	Rho*ELX+Z	1	LL	1		
26	ASCE 5 (c)	1	Y		DL	1.2	Sds*DL	0.2	Rho*ELX-Z	1	LL	1		
27	ASCE 5 (d)	1	Y		DL	1.2	Sds*DL	0.2	Rho*ELZ	1	LL	1		
28	ASCE 5 (e)	1	Y		DL	1.2	Sds*DL	0.2	Rho*ELZ+X	1	LL	1		
29	ASCE 5 (f)	1	Y		DL	1.2	Sds*DL	0.2	Rho*ELZ-X	1	LL	1		
30	ASCE 5 (g)	1	Y		DL	1.2	Sds*DL	0.2	Rho*ELX	-1	LL	1		
31	ASCE 5 (h)	1	Y		DL	1.2	Sds*DL	0.2	Rho*ELX+Z	-1	LL	1		
32	ASCE 5 (i)	1	Y		DL	1.2	Sds*DL	0.2	Rho*ELX-Z	-1	LL	1		
33	ASCE 5 (j)	1	Y		DL	1.2	Sds*DL	0.2	Rho*ELZ	-1	LL	1		
34	ASCE 5 (k)	1	Y		DL	1.2	Sds*DL	0.2	Rho*ELZ+X	-1	LL	1		
35	ASCE 5 (l)	1	Y		DL	1.2	Sds*DL	0.2	Rho*ELZ-X	-1	LL	1		
36	ASCE 7 (a)	1	Y		DL	0.9	Sds*DL	-0.2	Rho*ELX	1				
37	ASCE 7 (b)	1	Y		DL	0.9	Sds*DL	-0.2	Rho*ELX+Z	1				
38	ASCE 7 (c)	1	Y		DL	0.9	Sds*DL	-0.2	Rho*ELX-Z	1				
39	ASCE 7 (d)	1	Y		DL	0.9	Sds*DL	-0.2	Rho*ELZ	1				
40	ASCE 7 (e)	1	Y		DL	0.9	Sds*DL	-0.2	Rho*ELZ+X	1				
41	ASCE 7 (f)	1	Y		DL	0.9	Sds*DL	-0.2	Rho*ELZ-X	1				
42	ASCE 7 (g)	1	Y		DL	0.9	Sds*DL	-0.2	Rho*ELX	-1				
43	ASCE 7 (h)	1	Y		DL	0.9	Sds*DL	-0.2	Rho*ELX+Z	-1				
44	ASCE 7 (i)	1	Y		DL	0.9	Sds*DL	-0.2	Rho*ELX-Z	-1				
45	ASCE 7 (j)	1	Y		DL	0.9	Sds*DL	-0.2	Rho*ELZ	-1				
46	ASCE 7 (k)	1	Y		DL	0.9	Sds*DL	-0.2	Rho*ELZ+X	-1				
47	ASCE 7 (l)	1	Y		DL	0.9	Sds*DL	-0.2	Rho*ELZ-X	-1				

LOAD COMBINATIONS
per ASCE 7-05. Refer to Notes for comments

	Description	Solve	P-Delta	SRSS	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
	SKIP LOADING													
	DL + LL		applied only to cantilever portion											
50	ASCE 1		Y		DL	1.4								
51	ASCE 2 (a)		Y		DL	1.2	OL1	1.6	OL2	0.5	EPL	1.6		
52	ASCE 2 (b)		Y		DL	1.2	OL1	1.6	EPL	1.6				
53	ASCE 3 (a)		Y		DL	1.2	OL2	1.6	OL1	1				
	Wind Comb													
56	ASCE 3 (b) (a)		Y		DL	1.2	OL2	1.6	WLX	0.8	WLX+R	0.8		
57	ASCE 3 (b) (b)		Y		DL	1.2	OL2	1.6	WLZ	0.8	WLZ+R	0.8		
58	ASCE 3 (d) (a)		Y		DL	1.2	WLX	0.8	WLX+R	0.8				
59	ASCE 3 (d) (b)		Y		DL	1.2	WLZ	0.8	WLZ+R	0.8				
60	ASCE 4 (a) (a)		Y		DL	1.2	WLX	1.6	OL1	1	OL2	0.5	WLX+R	1.6
61	ASCE 4 (a) (b)		Y		DL	1.2	WLZ	1.6	OL1	1	OL2	0.5	WLZ+R	1.6
62	ASCE 4 (b) (a)		Y		DL	1.2	WLX	1.6	OL1	1	WLX+R	1.6		
63	ASCE 4 (b) (b)		Y		DL	1.2	WLZ	1.6	OL1	1	WLZ+R	1.6		
64	ASCE 4 (c) (a)		Y		DL	1.2	WLX	1.6	OL1	1			WLX+R	1.6
65	ASCE 4 (c) (b)		Y		DL	1.2	WLZ	1.6	OL1	1			WLZ+R	1.6
66	ASCE 6 (a)		Y		DL	0.9	WLX	1.6	WLX+R	1.6				
67	ASCE 6 (b)		Y		DL	0.9	WLZ	1.6	WLZ+R	1.6				

LOAD COMBINATIONS
per ASCE 7-05. Refer to Notes for comments

	Description	Solve	P-Delta	SRSS	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
	Eqk Comb													
70	ASCE 5 (a)		Y		DL	1.2	Sds*DL	0.2	Rho*ELX	1	OL1	1		
71	ASCE 5 (b)		Y		DL	1.2	Sds*DL	0.2	Rho*ELX+Z	1	OL1	1		
72	ASCE 5 (c)		Y		DL	1.2	Sds*DL	0.2	Rho*ELX-Z	1	OL1	1		
73	ASCE 5 (d)		Y		DL	1.2	Sds*DL	0.2	Rho*ELZ	1	OL1	1		
74	ASCE 5 (e)		Y		DL	1.2	Sds*DL	0.2	Rho*ELZ+X	1	OL1	1		
75	ASCE 5 (f)		Y		DL	1.2	Sds*DL	0.2	Rho*ELZ-X	1	OL1	1		
76	ASCE 5 (g)		Y		DL	1.2	Sds*DL	0.2	Rho*ELX	-1	OL1	1		
77	ASCE 5 (h)		Y		DL	1.2	Sds*DL	0.2	Rho*ELX+Z	-1	OL1	1		
78	ASCE 5 (i)		Y		DL	1.2	Sds*DL	0.2	Rho*ELX-Z	-1	OL1	1		
79	ASCE 5 (j)		Y		DL	1.2	Sds*DL	0.2	Rho*ELZ	-1	OL1	1		
80	ASCE 5 (k)		Y		DL	1.2	Sds*DL	0.2	Rho*ELZ+X	-1	OL1	1		
81	ASCE 5 (l)		Y		DL	1.2	Sds*DL	0.2	Rho*ELZ-X	-1	OL1	1		
	Service Check													
	Uniform LL													
85	ASCE 1		Y		DL	1								
86	ASCE 2		Y		DL	1	LL	1	EPL	1				
87	ASCE 3 (a)		Y		DL	1	RLL	1	EPL	1				
88	ASCE 4 (a)		Y		DL	1	LL	1	RLL	1	EPL	1		
	Skip LL, Cant													
91	ASCE 1		Y		DL	1								
92	ASCE 2		Y		DL	1	LL	1	EPL	1				
93	ASCE 3		Y		DL	1	RLL	1	EPL	1				
94	ASCE 4 (a)		Y		DL	1	LL	1	RLL	1	EPL	1		

Notes on Load Combinations:

1. Load combinations 2 through 81 are strength combinations used to check member capacities. Factors applied conform to 7-05 §2.3.2.
2. Load combinations 85 through 94 are un-factored service loads used to check for member deflection limits.
3. Load combinations conform to ASCE 7-05 & CBC 2010, exc LC 23 (used for purposes of calculating seismic mass).
4. P-delta analysis performed throughout in conformance with AISC 360-05.

CODE CHECK, STRENGTH
VERTICAL LOAD COMBINATIONS, FULL UNIFORM

Member	Shape	Code Chk	Loc	LC	V Chk	Loc	Dir	LC	Pnc/Ω	Pnt/Ω	Mnyy/Ω	Mnzz/Ω	Cb	Eqn
M121A	HSS6X6X8	0.757	0	3	0.007	0	y	2	154.065	244.958	41.497	41.497	2.25	H1-1a
M123A	HSS6X6X8	0.684	9.91	3	0.011	0	y	3	154.065	244.958	41.497	41.497	1.114	H1-1a
M85	HSS6X6X8	0.666	0	3	0.014	0	y	2	166.568	244.958	41.497	41.497	2.468	H1-1a
M122A	HSS6X6X8	0.635	8	3	0.047	0	y	3	218.534	244.958	41.497	41.497	2.268	H1-1a
M82	HSS6X6X8	0.619	14.705	3	0.012	14.705	y	2	166.568	244.958	41.497	41.497	2.99	H1-1a
M19	HSS8X8X8	0.481	0	3	0.085	0	z	3	344.786	371.856	86.078	86.078	2.168	H1-1b
M98	HSS8X8X8	0.448	0	3	0.133	0	z	3	368.372	371.856	86.078	86.078	2.231	H1-1b
M93	HSS6X6X8	0.429	0	3	0.041	19.805	y	3	121.694	244.958	41.497	41.497	1.843	H1-1a
M23	Box 8x8 w/ 1/2"	0.353	8.5	3	0.059	0	z	5	826.295	881.437	221.507	221.507	2.244	H1-1b
M22	Box 8x8 w/ 1/2"	0.349	0	3	0.082	8.578	y	3	748.893	881.437	221.507	221.507	2.49	H1-1b
M13	W10X54	0.347	8	3	0.16	0.5	y	3	426.598	473.054	78.094	166.168	2.184	H1-1b
M17	Box 8x8 w/ 1/2"	0.343	0	3	0.063	0	z	3	690.988	881.437	221.507	221.507	2.08	H1-1b
M18	Box 8x8 w/ 1/2"	0.33	0	3	0.074	8.578	y	4	748.893	881.437	221.507	221.507	2.729	H1-1b
M89	HSS6X6X8	0.323	0	3	0.012	0	y	2	181.213	244.958	41.497	41.497	2.572	H1-1a
M20	HSS8X8X8	0.275	0	3	0.085	0	z	3	319.773	371.856	86.078	86.078	2.759	H1-1b
M21	HSS8X8X8	0.264	6.274	3	0.07	0	z	3	289.745	371.856	86.078	86.078	2.037	H1-1b
M74	Box 8x8 w/ 1/2"	0.256	0	5	0.042	0	y	3	826.295	881.437	221.507	221.507	2.238	H1-1b
M43	W10X54	0.251	12.25	5	0.12	12.25	y	5	137.132	473.054	78.094	166.168	2.016	H1-1b
M8A	W10X54	0.246	12.25	3	0.094	12.25	y	3	60.948	473.054	78.094	166.168	2.907	H1-1b
M24	Box 8x8 w/ 1/2"	0.24	8.5	3	0.08	7.438	y	3	826.295	881.437	221.507	221.507	3	H1-1b
M30	Box 8x8 w/ 1/2"	0.221	0	3	0.051	0	y	5	832.413	881.437	221.507	221.507	2.572	H1-1b
M97A	W10X54	0.206	0	5	0.19	1.313	y	3	466.227	473.054	78.094	166.168	2.161	H1-1b
M36	W10X54	0.202	25.729	5	0.102	0	y	5	74.454	473.054	78.094	166.168	2.747	H1-1b
M76	W10X26	0.181	0	3	0.012	0	y	5	42.633	164.048	13.473	46.944	1.309	H1-1b
M2	W10X54	0.176	1.203	3	0.224	0.401	y	3	72.533	473.054	78.094	166.168	2.02	H1-1b
M1	W10X54	0.174	1.203	3	0.567	0.401	y	3	72.533	473.054	78.094	166.168	3	H1-1b
M42	W10X54	0.172	0	5	0.072	1.896	y	5	344.691	473.054	78.094	166.168	3	H1-1b
M33	Box 8x8 w/ 1/2"	0.171	11	5	0.058	8.021	y	5	791.051	881.437	221.507	221.507	2.805	H1-1b
M32	Box 8x8 w/ 1/2"	0.171	0	5	0.051	0	y	3	832.413	881.437	221.507	221.507	2.509	H1-1b
M31	Box 8x8 w/ 1/2"	0.17	0	3	0.086	0	y	3	791.051	881.437	221.507	221.507	3	H1-1b
M6	W10X54	0.169	8	3	0.024	4	y	3	426.598	473.054	78.094	166.168	1.332	H1-1b
M47	Box 8x8 w/ 1/2"	0.169	8.422	3	0.03	5.328	z	3	690.988	881.437	221.507	221.507	1.841	H1-1b

CODE CHECK, STRENGTH
VERTICAL LOAD COMBINATIONS, FULL UNIFORM

Member	Shape	Code Chk	Loc	LC	V Chk	Loc	Dir	LC	Pnc/Ω	Pnt/Ω	Mnyy/Ω	Mnzz/Ω	Cb	Eqn
M73A	W10X26	0.157	13.935	4	0.042	16.257	y	3	42.633	164.048	13.473	38.347	1.07	H1-1b
M55	W10X54	0.153	0	5	0.058	0	y	5	266.963	473.054	78.094	166.168	1.514	H1-1b
M100A	W10X54	0.153	18.58	3	0.03	18.58	y	3	270.872	473.054	78.094	166.168	2.452	H1-1b
M11	W10X54	0.146	0	3	0.041	0	y	3	270.872	473.054	78.094	166.168	2.226	H1-1b
M100	W10X54	0.145	0	5	0.119	0	y	3	456.681	473.054	78.094	166.168	2.224	H1-1b
M41	W10X54	0.142	18.821	5	0.057	18.821	y	5	266.963	473.054	78.094	166.168	2.143	H1-1b
M9	W10X54	0.139	0	5	0.063	28	y	2	60.948	473.054	78.094	166.168	2.864	H1-1b
M46	W10X54	0.138	0	5	0.071	0	y	5	266.963	473.054	78.094	166.168	2.467	H1-1b
M99	HSS8X8X8	0.133	0	3	0.071	0	z	3	368.372	371.856	86.078	86.078	1.122	H1-1b
M16	Box 8x8 w/ 1/2"	0.132	0	5	0.017	0	y	3	826.295	881.437	221.507	221.507	2.025	H1-1b
M35	W10X54	0.127	13.854	5	0.09	13.854	y	5	74.454	473.054	78.094	166.168	2.859	H1-1b
M75	Box 8x8 w/ 1/2"	0.121	8.5	3	0.013	0	y	2	826.295	881.437	221.507	221.507	1.377	H1-1b
M99A	W10X54	0.114	0	3	0.033	0	y	2	395.893	473.054	78.094	166.168	2.323	H1-1b
M51	W10X54	0.111	10.547	5	0.102	3.164	y	5	337.411	473.054	78.094	166.168	1.28	H1-1b
M4	W10X54	0.11	18.58	3	0.016	2.322	y	3	270.872	473.054	78.094	157.604	1.107	H1-1b
M34	Box 8x8 w/ 1/2"	0.109	5	5	0.036	5	z	5	832.413	881.437	221.507	221.507	1.67	H1-1b
M98A	W10X54	0.107	0	3	0.041	0	y	3	395.893	473.054	78.094	166.168	2.336	H1-1b
M15	Box 8x8 w/ 1/2"	0.105	8.5	3	0.013	0	y	3	826.295	881.437	221.507	221.507	1.916	H1-1b
M101	Box 8x8 w/ 1/2"	0.101	8.594	5	0.019	0	z	5	690.988	881.437	221.507	221.507	2.507	H1-1b
M39	Box 8x8 w/ 1/2"	0.092	7.906	5	0.02	8.021	z	5	791.051	881.437	221.507	221.507	2.002	H1-1b
M86	W10X54	0.089	35	5	0.035	35	y	5	87.765	473.054	78.094	166.168	2.708	H1-1b
M3	W10X54	0.089	10.838	3	0.013	0	y	3	270.872	473.054	78.094	151.914	1.067	H1-1b
M28	W10X54	0.088	18	3	0.035	18	y	3	280.315	473.054	78.094	166.168	1.484	H1-1b
M26	W10X54	0.083	13.854	2	0.054	13.854	y	2	74.454	473.054	78.094	166.168	2.542	H1-1b
M38	W10X54	0.083	0	5	0.05	0	y	5	266.963	473.054	78.094	166.168	2.428	H1-1b
M27	W10X54	0.083	12	2	0.046	12	y	2	374.89	473.054	78.094	166.168	3	H1-1b
M5	W10X54	0.082	10.994	3	0.012	8.944	y	2	282.132	473.054	78.094	166.168	1.16	H1-1b
M25	W10X54	0.077	0	2	0.057	0	y	2	270.872	473.054	78.094	166.168	2.114	H1-1b
M124A	W10X54	0.072	0	5	0.073	0	y	5	344.691	473.054	78.094	166.168	1.38	H1-1b
M45	W10X54	0.072	19.058	5	0.024	19.058	y	5	263.111	473.054	78.094	166.168	2.098	H1-1b
M29	W10X54	0.072	2.292	5	0.179	0.917	y	5	389.078	473.054	78.094	166.168	1.231	H1-1b
M91	W10X54	0.072	12	5	0.029	12	y	5	374.89	473.054	78.094	166.168	2.401	H1-1b

CODE CHECK, STRENGTH
VERTICAL LOAD COMBINATIONS, FULL UNIFORM

Member	Shape	Code Chk	Loc	LC	V Chk	Loc	Dir	LC	Pnc/Ω	Pnt/Ω	Mnyy/Ω	Mnzz/Ω	Cb	Eqn
M40	W10X54	0.069	0	5	0.028	0	y	5	266.963	473.054	78.094	166.168	2.388	H1-1b
M12	W10X54	0.068	0	3	0.023	2.981	y	3	282.132	473.054	78.094	166.168	2.032	H1-1b
M48	W10X54	0.067	15.58	5	0.032	15.58	y	5	319.63	473.054	78.094	166.168	2.676	H1-1b
M44	W10X54	0.054	0	5	0.029	0	y	5	266.963	473.054	78.094	166.168	2.829	H1-1b
M37	W10X54	0.051	18.821	5	0.019	18.821	y	5	266.963	473.054	78.094	166.168	3	H1-1b
M10	W10X54	0.033	0	3	0.016	0	y	2	270.872	473.054	78.094	166.168	1.858	H1-1b
M49	W10X54	0.03	0	3	0.021	0.262	y	3	366.357	473.054	78.094	166.168	3	H1-1b
M8	W10X54	0.025	18.58	3	0.03	13.935	y	3	270.872	473.054	78.094	166.168	1.222	H1-1b
M124B	W10X54	0.023	23	2	0.014	23	y	2	201.304	473.054	78.094	166.168	1.435	H1-1b
M70	W10X54	0.02	9.29	2	0.009	18.58	y	2	270.872	473.054	78.094	161.808	1.137	H1-1b
M77	Box 8x8 w/ 1/2"	0.001	10.5	2	0	0	z	3	798.691	881.437	221.507	221.507	1.667	H1-1b

Notes:
1. Code check evaluates demand against code prescribed capacity, and is noted as a ratio. If check is less than 1.0 (or as permitted by code), then the member is acceptable.
2. Loc: Location along member that controls design.
3. LC: Load combination per ASCE 7-05..
4. V Chk: Shear check expressed as a ratio of demand against code prescribed capacity..
5. Dir: Direction controlling force is applied. Lower case indicates local coordinate sytem.
6. Design Capacities divided by the Omega safety factors: Pnc/Ω, Pnt/Ω, Mnyy/Ω, Mnzz/Ω.
7. Cb: Flexural Bending coefficient per AISC 360-05 Specification.

CODE CHECK, STRENGTH
VERTICAL LOAD COMBINATIONS, FULL UNIFORM LOADING

Conclusion: Maximum ratio of demand against member capacity = 0.76, less than 1.0.
Members are adequate per DL + LL combinations.

Skip loading same combinations does not lead to appreciable difference in system response. Therefore, results not included.

CODE CHECK, STRENGTH
LOAD COMBINATIONS incl WIND, SKIP UNIFORM LOADING

Member	Shape	Code Chk	Loc	LC	V Chk	Loc	Dir	LC	Pnc/Ω	Pnt/Ω	Mnyy/Ω	Mnzz/Ω	Cb	Eqn
M121A	HSS6X6X8	0.8	0	61	0.016	0	y	61	154.065	244.958	41.497	41.497	2.135	H1-1a
M43	W10X54	0.776	28	60	0.062	28	y	57	137.132	473.054	78.094	166.168	2.739	H1-1a
M20	HSS8X8X8	0.75	12.01	60	0.178	6.38	y	61	319.773	371.856	86.078	86.078	3	H1-1b
M100	W10X54	0.683	4.67	60	0.181	3.697	y	60	456.681	473.054	78.094	166.168	2.204	H1-1a
M123A	HSS6X6X8	0.672	12.597	61	0.014	0	y	61	154.065	244.958	41.497	41.497	1.268	H1-1a
M18	Box 8x8 w/ 1/2"	0.664	13.5	60	0.207	8.578	y	60	748.893	881.437	221.507	221.507	3	H1-1b
M122A	HSS6X6X8	0.636	0	61	0.046	0	y	61	218.534	244.958	41.497	41.497	2.255	H1-1a
M82	HSS6X6X8	0.601	14.705	61	0.011	14.705	y	60	166.568	244.958	41.497	41.497	2.733	H1-1a
M32	Box 8x8 w/ 1/2"	0.453	8	61	0.187	5	y	61	832.413	881.437	221.507	221.507	3	H1-1b
M85	HSS6X6X8	0.437	0	60	0.02	0	y	61	166.568	244.958	41.497	41.497	2.51	H1-1a
M98	HSS8X8X8	0.409	0	61	0.105	0	z	61	368.372	371.856	86.078	86.078	2.202	H1-1b
M34	Box 8x8 w/ 1/2"	0.405	8	60	0.205	5	y	60	832.413	881.437	221.507	221.507	3	H1-1b
M30	Box 8x8 w/ 1/2"	0.373	0	60	0.109	0	y	60	832.413	881.437	221.507	221.507	2.101	H1-1b
M21	HSS8X8X8	0.357	15.443	61	0.065	6.435	y	61	289.745	371.856	86.078	86.078	2.285	H1-1b
M17	Box 8x8 w/ 1/2"	0.338	0	61	0.059	0	z	61	690.988	881.437	221.507	221.507	2.099	H1-1b
M89	HSS6X6X8	0.311	0	61	0.023	0	y	61	181.213	244.958	41.497	41.497	2.028	H1-1a
M97A	W10X54	0.304	0	61	0.209	0.719	y	61	466.227	473.054	78.094	166.168	1.607	H1-1b
M22	Box 8x8 w/ 1/2"	0.293	0	61	0.104	8.578	y	60	748.893	881.437	221.507	221.507	2.947	H1-1b
M19	HSS8X8X8	0.269	8.5	60	0.05	0	z	60	344.786	371.856	86.078	86.078	1.964	H1-1b
M74	Box 8x8 w/ 1/2"	0.257	8.5	61	0.042	0	y	61	826.295	881.437	221.507	221.507	2.222	H1-1b
M16	Box 8x8 w/ 1/2"	0.257	0	61	0.026	0	z	61	826.295	881.437	221.507	221.507	2.159	H1-1b
M39	Box 8x8 w/ 1/2"	0.25	8.021	61	0.115	8.021	y	61	791.051	881.437	221.507	221.507	2.231	H1-1b
M23	Box 8x8 w/ 1/2"	0.249	0	60	0.041	0	z	60	826.295	881.437	221.507	221.507	2.272	H1-1b
M137	W10X54	0.238	4.67	60	0.181	3.697	y	60	456.681	473.054	78.094	166.168	2.204	H1-1b
M31	Box 8x8 w/ 1/2"	0.21	0	60	0.101	0	y	60	791.051	881.437	221.507	221.507	3	H1-1b
M8A	W10X54	0.199	12.25	60	0.119	28	y	60	60.948	473.054	78.094	166.168	2.662	H1-1b
M42	W10X54	0.198	0	57	0.095	0	y	57	344.691	473.054	78.094	166.168	2.886	H1-1b
M33	Box 8x8 w/ 1/2"	0.194	0	61	0.035	0	y	60	791.051	881.437	221.507	221.507	2.332	H1-1b
M75	Box 8x8 w/ 1/2"	0.192	8.5	61	0.045	0	z	61	826.295	881.437	221.507	221.507	1.113	H1-1b
M99	HSS8X8X8	0.19	0	60	0.058	0	z	60	368.372	371.856	86.078	86.078	1.626	H1-1b
M9	W10X54	0.185	28	61	0.111	28	y	61	60.948	473.054	78.094	166.168	3	H1-1b
M24	Box 8x8 w/ 1/2"	0.185	8.5	60	0.08	7.438	y	60	826.295	881.437	221.507	221.507	3	H1-1b

CODE CHECK, STRENGTH
LOAD COMBINATIONS incl WIND, SKIP UNIFORM LOADING

Member	Shape	Code Chk	Loc	LC	V Chk	Loc	Dir	LC	Pnc/Ω	Pnt/Ω	Mnyy/Ω	Mnzz/Ω	Cb	Eqn
M15	Box 8x8 w/ 1/2"	0.184	0	61	0.02	0	z	63	826.295	881.437	221.507	221.507	2.159	H1-1b
M13	W10X54	0.179	8	60	0.088	2.667	y	60	426.598	473.054	78.094	166.168	2.182	H1-1b
M48	W10X54	0.175	15.58	61	0.082	15.418	y	61	319.63	473.054	78.094	166.168	2.915	H1-1b
M2	W10X54	0.175	1.203	61	0.317	0.401	y	61	72.533	473.054	78.094	166.168	2.063	H1-1b
M93	HSS6X6X8	0.173	19.805	60	0.027	19.805	y	60	121.694	244.958	41.497	41.497	2.039	H1-1b
M135	W10X54	0.164	0	60	0.076	0	y	56	137.132	473.054	78.094	166.168	3	H1-1b
M101	Box 8x8 w/ 1/2"	0.157	16.5	61	0.034	8.594	y	61	690.988	881.437	221.507	221.507	3	H1-1b
M46	W10X54	0.153	0	60	0.058	0.392	y	60	266.963	473.054	78.094	166.168	2.841	H1-1b
M1	W10X54	0.153	1.203	61	0.601	0.401	y	61	72.533	473.054	78.094	166.168	3	H1-1b
M98A	W10X54	0.138	0	61	0.055	0	y	61	395.893	473.054	78.094	166.168	2.312	H1-1b
M91	W10X54	0.136	12	61	0.048	12	y	61	374.89	473.054	78.094	166.168	2.176	H1-1b
M73A	W10X26	0.132	16.257	61	0.032	16.257	y	60	42.633	164.048	13.473	56.228	1.591	H1-1b
M11	W10X54	0.131	0	61	0.037	0	y	61	270.872	473.054	78.094	166.168	2.24	H1-1b
M47	Box 8x8 w/ 1/2"	0.119	8.422	60	0.058	16.328	y	61	690.988	881.437	221.507	221.507	2.281	H1-1b
M100A	W10X54	0.118	18.58	61	0.025	18.58	y	61	270.872	473.054	78.094	166.168	2.458	H1-1b
M36	W10X54	0.118	0	59	0.064	0	y	63	74.454	473.054	78.094	166.168	2.433	H1-1b
M76	W10X26	0.114	0	60	0.009	0	y	60	42.633	164.048	13.473	40.98	1.143	H1-1b
M55	W10X54	0.112	0	57	0.066	0	y	56	266.963	473.054	78.094	166.168	1.986	H1-1b
M26	W10X54	0.1	13.854	60	0.055	13.854	y	61	74.454	473.054	78.094	166.168	2.224	H1-1b
M41	W10X54	0.097	18.821	57	0.052	18.821	y	57	266.963	473.054	78.094	166.168	2.256	H1-1b
M124A	W10X54	0.091	14	56	0.08	0	y	57	344.691	473.054	78.094	166.168	2.341	H1-1b
M29	W10X54	0.088	0.917	63	0.196	0.917	y	61	389.078	473.054	78.094	166.168	1.472	H1-1b
M51	W10X54	0.087	10.547	61	0.076	3.164	y	58	337.411	473.054	78.094	166.168	1.329	H1-1b
M4	W10X54	0.087	7.355	63	0.014	0.387	y	63	270.872	473.054	78.094	146.008	1.026	H1-1b
M6	W10X54	0.079	8	61	0.014	4	y	61	426.598	473.054	78.094	166.168	1.377	H1-1b
M3	W10X54	0.072	10.451	61	0.011	0	y	61	270.872	473.054	78.094	151.858	1.067	H1-1b
M35	W10X54	0.071	26.125	60	0.085	26.125	y	60	74.454	473.054	78.094	166.168	2.291	H1-1b
M121B	W10X54	0.07	26.125	60	0.085	26.125	y	60	74.454	473.054	78.094	166.168	2.291	H1-1b
M86	W10X54	0.07	35	61	0.023	35	y	61	87.765	473.054	78.094	166.168	3	H1-1b
M28	W10X54	0.068	18	60	0.027	18	y	60	280.315	473.054	78.094	166.168	1.563	H1-1b
M27	W10X54	0.068	12	61	0.036	12	y	63	374.89	473.054	78.094	166.168	2.666	H1-1b
M25	W10X54	0.068	0	63	0.049	0	y	60	270.872	473.054	78.094	166.168	2.353	H1-1b

CODE CHECK, STRENGTH
LOAD COMBINATIONS incl WIND, SKIP UNIFORM LOADING

Member	Shape	Code Chk	Loc	LC	V Chk	Loc	Dir	LC	Pnc/Ω	Pnt/Ω	Mnyy/Ω	Mnzz/Ω	Cb	Eqn
M38	W10X54	0.065	0	63	0.035	0	y	63	266.963	473.054	78.094	166.168	2.377	H1-1b
M40	W10X54	0.063	18.821	57	0.034	18.821	y	57	266.963	473.054	78.094	166.168	2.093	H1-1b
M5	W10X54	0.055	11.367	61	0.009	8.944	y	60	282.132	473.054	78.094	150.732	1.046	H1-1b
M12	W10X54	0.054	0	60	0.018	1.118	y	60	282.132	473.054	78.094	166.168	2.333	H1-1b
M45	W10X54	0.053	0	61	0.012	6.948	y	58	263.111	473.054	78.094	166.168	2.138	H1-1b
M99A	W10X54	0.049	0	61	0.013	0	y	58	395.893	473.054	78.094	166.168	2.572	H1-1b
M8	W10X54	0.047	16.257	61	0.028	16.257	y	60	270.872	473.054	78.094	166.168	1.556	H1-1b
M136	W10X54	0.032	11.523	61	0.033	11.523	y	61	319.63	473.054	78.094	166.168	1.852	H1-1b
M44	W10X54	0.029	0	61	0.02	18.821	y	60	266.963	473.054	78.094	166.168	2.381	H1-1b
M49	W10X54	0.028	12.58	60	0.034	11.663	y	61	366.357	473.054	78.094	166.168	3	H1-1b
M10	W10X54	0.028	0	61	0.013	0	y	61	270.872	473.054	78.094	166.168	1.746	H1-1b
M124B	W10X54	0.021	23	58	0.012	23	y	59	201.304	473.054	78.094	166.168	1.488	H1-1b
M37	W10X54	0.02	0	60	0.014	18.821	y	63	266.963	473.054	78.094	166.168	2.381	H1-1b
M70	W10X54	0.017	9.484	61	0.008	0	y	60	270.872	473.054	78.094	161.804	1.137	H1-1b
M151	W10X54	0.013	18.58	61	0.008	18.58	y	63	270.872	473.054	78.094	166.168	2.318	H1-1b
M150	W10X54	0.012	18.821	59	0.008	18.821	y	62	266.963	473.054	78.094	166.168	2.191	H1-1b
M153	W10X54	0.012	0	60	0.008	0	y	60	270.872	473.054	78.094	166.168	2.312	H1-1b
M152	W10X54	0.009	9.133	61	0.008	18.655	y	56	269.655	473.054	78.094	166.168	1.275	H1-1b
M77	Box 8x8 w/ 1/2"	0.001	10.5	60	0	0	z	60	798.691	881.437	221.507	221.507	1.667	H1-1b

CODE CHECK, STRENGTH
LOAD COMBINATIONS incl WIND, SKIP UNIFORM LOADING

Notes:

1. Code check evaluates demand against code prescribed capacity, and is noted as a ratio. If check is less than 1.Ø (or as permitted by code), then the member is acceptable.
2. Loc: Location along member that controls design.
3. LC: Load combination per ASCE 7-Ø5..
4. V Chk: Shear check expressed as a ratio of demand against code prescribed capacity..
5. Dir: Direction controlling force is applied. Lower case indicates local coordinate sytem.
6. Design Capacities divided by the Omega safety factors: Pnc/Ω, Pnt/Ω, Mnyy/Ω, Mnzz/Ω.
7. Cb: Flexural Bending coefficient per AISC 360-05 Specification.

CODE CHECK, STRENGTH
LOAD COMBINATIONS incl WIND, SKIP UNIFORM LIVE LOADING

Conclusion: Maximum ratio of demand against member capacity = Ø.8Ø, less than 1.Ø.
Members are adequate per DL + LL + W combinations.

Skip loading leads to maximum stress ratios for WIND combinations. Same combinations with full live load do not control.

CODE CHECK, STRENGTH
LOAD COMBINATIONS incl SEISMIC, SKIP UNIFORM LOADING

Member	Shape	Code Chk	Loc	LC	V Chk	Loc	Dir	LC	Pnc/Ω	Pnt/Ω	Mnyy/Ω	Mnzz/Ω	Cb	Eqn
M121A	HSS6X6X8	0.976	0	74	0.028	0	y	74	154.065	244.958	41.497	41.497	2.095	H1-1a
M20	HSS8X8X8	0.97	12.01	80	0.246	6.38	y	80	319.773	371.856	86.078	86.078	3	H1-1b
M43	W10X54	0.962	28	81	0.064	3.792	y	72	137.132	473.054	78.094	166.168	2.33	H1-1a
M100	W10X54	0.849	4.67	81	0.229	3.697	y	81	456.681	473.054	78.094	166.168	2.205	H1-1a
M18	Box 8x8 w/ 1/2"	0.839	13.5	80	0.27	8.578	y	80	748.893	881.437	221.507	221.507	3	H1-1b
M30	Box 8x8 w/ 1/2"	0.818	0	81	0.208	0	z	81	832.413	881.437	221.507	221.507	2.091	H1-1b
M123A	HSS6X6X8	0.813	13.437	74	0.019	0	y	74	154.065	244.958	41.497	41.497	1.327	H1-1a
M82	HSS6X6X8	0.79	14.705	72	0.02	7.353	z	80	166.568	244.958	41.497	41.497	2.587	H1-1a
M122A	HSS6X6X8	0.761	0	74	0.054	0	y	74	218.534	244.958	41.497	41.497	2.252	H1-1a
M23	Box 8x8 w/ 1/2"	0.649	8.5	81	0.127	0	z	81	826.295	881.437	221.507	221.507	2.27	H1-1b
M32	Box 8x8 w/ 1/2"	0.648	8	74	0.246	5	y	72	832.413	881.437	221.507	221.507	3	H1-1b
M85	HSS6X6X8	0.61	0	80	0.034	0	y	74	166.568	244.958	41.497	41.497	2.489	H1-1a
M22	Box 8x8 w/ 1/2"	0.576	13.5	80	0.138	8.578	y	80	748.893	881.437	221.507	221.507	3	H1-1b
M24	Box 8x8 w/ 1/2"	0.562	8.5	81	0.145	7.438	z	81	826.295	881.437	221.507	221.507	3	H1-1b
M98	HSS8X8X8	0.525	0	74	0.151	0	z	74	368.372	371.856	86.078	86.078	2.199	H1-1b
M74	Box 8x8 w/ 1/2"	0.516	8.5	81	0.097	0	z	81	826.295	881.437	221.507	221.507	2.218	H1-1b
M34	Box 8x8 w/ 1/2"	0.51	8	72	0.26	5	y	72	832.413	881.437	221.507	221.507	3	H1-1b
M31	Box 8x8 w/ 1/2"	0.486	0	81	0.138	0	y	71	791.051	881.437	221.507	221.507	3	H1-1b
M17	Box 8x8 w/ 1/2"	0.465	0	74	0.083	0	z	74	690.988	881.437	221.507	221.507	2.119	H1-1b
M21	HSS8X8X8	0.457	15.443	74	0.096	0	z	74	289.745	371.856	86.078	86.078	2.286	H1-1b
M16	Box 8x8 w/ 1/2"	0.431	0	74	0.04	0	z	74	826.295	881.437	221.507	221.507	2.155	H1-1a
M89	HSS6X6X8	0.425	0	72	0.034	0	y	74	181.213	244.958	41.497	41.497	2.033	H1-1a
M19	HSS8X8X8	0.416	0	80	0.07	0	z	80	344.786	371.856	86.078	86.078	2.226	H1-1b
M75	Box 8x8 w/ 1/2"	0.398	8.5	80	0.087	0	z	74	826.295	881.437	221.507	221.507	1.759	H1-1b
M15	Box 8x8 w/ 1/2"	0.392	0	74	0.043	0	z	81	826.295	881.437	221.507	221.507	2.159	H1-1a
M39	Box 8x8 w/ 1/2"	0.39	7.906	74	0.132	8.021	y	74	791.051	881.437	221.507	221.507	2.226	H1-1a
M97A	W10X54	0.369	0	74	0.253	1.188	y	74	466.227	473.054	78.094	166.168	1.622	H1-1b
M33	Box 8x8 w/ 1/2"	0.354	0	80	0.07	8.021	z	80	791.051	881.437	221.507	221.507	2.318	H1-1b
M99	HSS8X8X8	0.336	0	80	0.116	0	z	80	368.372	371.856	86.078	86.078	1.621	H1-1b
M93	HSS6X6X8	0.311	19.805	78	0.038	19.805	y	80	121.694	244.958	41.497	41.497	2.028	H1-1a
M46	W10X54	0.307	0	81	0.082	1.176	y	81	266.963	473.054	78.094	166.168	3	H1-1b
M137	W10X54	0.299	4.67	81	0.229	3.697	y	81	456.681	473.054	78.094	166.168	2.205	H1-1b

CODE CHECK, STRENGTH
LOAD COMBINATIONS incl SEISMIC, SKIP UNIFORM LOADING

Member	Shape	Code Chk	Loc	LC	V Chk	Loc	Dir	LC	Pnc/Ω	Pnt/Ω	Mnyy/Ω	Mnzz/Ω	Cb	Eqn
M76	W10X26	0.274	0	81	0.021	0	y	81	42.633	164.048	13.473	54.868	1.53	H1-1b
M8A	W10X54	0.267	12.25	80	0.155	28	y	80	60.948	473.054	78.094	166.168	2.673	H1-1b
M13	W10X54	0.255	8	80	0.127	2.667	y	80	426.598	473.054	78.094	166.168	2.195	H1-1b
M47	Box 8x8 w/ 1/2"	0.249	0	81	0.081	16.328	y	75	690.988	881.437	221.507	221.507	2.218	H1-1b
M101	Box 8x8 w/ 1/2"	0.244	16.5	74	0.047	8.594	y	72	690.988	881.437	221.507	221.507	3	H1-1b
M73A	W10X26	0.239	16.257	74	0.053	16.257	y	80	42.633	164.048	13.473	56.228	1.782	H1-1b
M48	W10X54	0.238	15.58	81	0.115	15.58	y	72	319.63	473.054	78.094	166.168	3	H1-1b
M9	W10X54	0.224	28	74	0.134	28	y	74	60.948	473.054	78.094	166.168	3	H1-1b
M42	W10X54	0.211	0	74	0.089	0	y	74	344.691	473.054	78.094	166.168	2.794	H1-1b
M11	W10X54	0.209	0	74	0.057	0	y	74	270.872	473.054	78.094	166.168	2.227	H1-1b
M2	W10X54	0.205	1.203	74	0.382	0.401	y	74	72.533	473.054	78.094	166.168	2.095	H1-1b
M135	W10X54	0.2	0	72	0.08	0	y	81	137.132	473.054	78.094	166.168	3	H1-1b
M100A	W10X54	0.186	18.58	74	0.038	18.58	y	74	270.872	473.054	78.094	166.168	2.403	H1-1b
M1	W10X54	0.18	1.203	72	0.718	0.401	y	72	72.533	473.054	78.094	166.168	3	H1-1b
M91	W10X54	0.179	0	72	0.064	12	y	72	374.89	473.054	78.094	166.168	2.185	H1-1b
M98A	W10X54	0.166	0	74	0.066	1.203	y	74	395.893	473.054	78.094	166.168	2.31	H1-1b
M29	W10X54	0.142	0.917	74	0.278	0.917	y	74	389.078	473.054	78.094	166.168	1.528	H1-1b
M36	W10X54	0.136	0	75	0.073	0	y	75	74.454	473.054	78.094	166.168	2.412	H1-1b
M26	W10X54	0.129	13.854	72	0.069	13.854	y	81	74.454	473.054	78.094	166.168	2.179	H1-1b
M51	W10X54	0.127	0	81	0.093	3.164	y	81	337.411	473.054	78.094	166.168	1.388	H1-1b
M41	W10X54	0.124	18.821	74	0.044	18.821	y	74	266.963	473.054	78.094	166.168	2.545	H1-1b
M55	W10X54	0.121	0	80	0.047	0	y	80	266.963	473.054	78.094	166.168	2.247	H1-1b
M4	W10X54	0.111	0	74	0.018	0	y	74	270.872	473.054	78.094	156.209	1.097	H1-1b
M6	W10X54	0.102	8	75	0.017	4	y	78	426.598	473.054	78.094	166.168	1.399	H1-1b
M8	W10X54	0.098	18.58	80	0.035	16.257	y	81	270.872	473.054	78.094	166.168	1.642	H1-1b
M35	W10X54	0.089	26.125	72	0.105	26.125	y	72	74.454	473.054	78.094	166.168	2.277	H1-1b
M27	W10X54	0.089	12	75	0.043	12	y	75	374.89	473.054	78.094	166.168	2.708	H1-1b
M121B	W10X54	0.087	26.125	72	0.105	26.125	y	72	74.454	473.054	78.094	166.168	2.277	H1-1b
M40	W10X54	0.086	18.821	74	0.034	18.821	y	74	266.963	473.054	78.094	166.168	1.822	H1-1b
M28	W10X54	0.085	18	71	0.033	18	y	80	280.315	473.054	78.094	166.168	1.633	H1-1b
M86	W10X54	0.084	35	75	0.028	35	y	71	87.765	473.054	78.094	166.168	3	H1-1b
M3	W10X54	0.082	10.838	74	0.013	0	y	74	270.872	473.054	78.094	154.124	1.083	H1-1b

CODE CHECK, STRENGTH
LOAD COMBINATIONS incl SEISMIC, SKIP UNIFORM LOADING

Member	Shape	Code Chk	Loc	LC	V Chk	Loc	Dir	LC	Pnc/Ω	Pnt/Ω	Mnyy/Ω	Mnzz/Ω	Cb	Eqn
M25	W10X54	0.08	18.58	75	0.056	0	y	81	270.872	473.054	78.094	166.168	2.562	H1-1b
M38	W10X54	0.076	0	74	0.041	0	y	74	266.963	473.054	78.094	166.168	2.309	H1-1b
M124A	W10X54	0.074	14	72	0.057	0	y	74	344.691	473.054	78.094	166.168	2.83	H1-1b
M45	W10X54	0.073	0	75	0.021	7.147	y	81	263.111	473.054	78.094	166.168	1.84	H1-1b
M5	W10X54	0.067	11.18	75	0.011	8.944	y	80	282.132	473.054	78.094	151.615	1.052	H1-1b
M99A	W10X54	0.066	0	75	0.019	0	y	78	395.893	473.054	78.094	166.168	2.495	H1-1b
M12	W10X54	0.066	0	80	0.023	2.236	y	80	282.132	473.054	78.094	166.168	2.088	H1-1b
M49	W10X54	0.064	12.58	81	0.048	11.663	y	75	366.357	473.054	78.094	166.168	3	H1-1b
M10	W10X54	0.046	0	74	0.019	0	y	74	270.872	473.054	78.094	166.168	1.853	H1-1b
M136	W10X54	0.045	11.523	72	0.046	11.523	y	72	319.63	473.054	78.094	166.168	1.753	H1-1b
M44	W10X54	0.033	0	77	0.023	18.821	y	71	266.963	473.054	78.094	166.168	2.381	H1-1b
M124B	W10X54	0.026	23	81	0.014	23	y	77	201.304	473.054	78.094	166.168	1.498	H1-1b
M37	W10X54	0.023	0	80	0.016	18.821	y	74	266.963	473.054	78.094	166.168	2.383	H1-1b
M70	W10X54	0.02	9.484	74	0.009	0	y	80	270.872	473.054	78.094	161.799	1.136	H1-1b
M152	W10X54	0.016	18.655	80	0.009	18.655	y	81	269.655	473.054	78.094	166.168	2.949	H1-1b
M151	W10X54	0.016	18.58	74	0.009	18.58	y	74	270.872	473.054	78.094	166.168	2.36	H1-1b
M150	W10X54	0.015	18.821	75	0.01	18.821	y	80	266.963	473.054	78.094	166.168	2.704	H1-1b
M153	W10X54	0.015	0	81	0.01	0	y	81	270.872	473.054	78.094	166.168	2.27	H1-1b
M77	Box 8x8 w/ 1/2"	0.001	10.5	81	0	0	z	80	798.691	881.437	221.507	221.507	1.667	H1-1b

CODE CHECK, STRENGTH
LOAD COMBINATIONS incl SEISMIC, SKIP UNIFORM LOADING

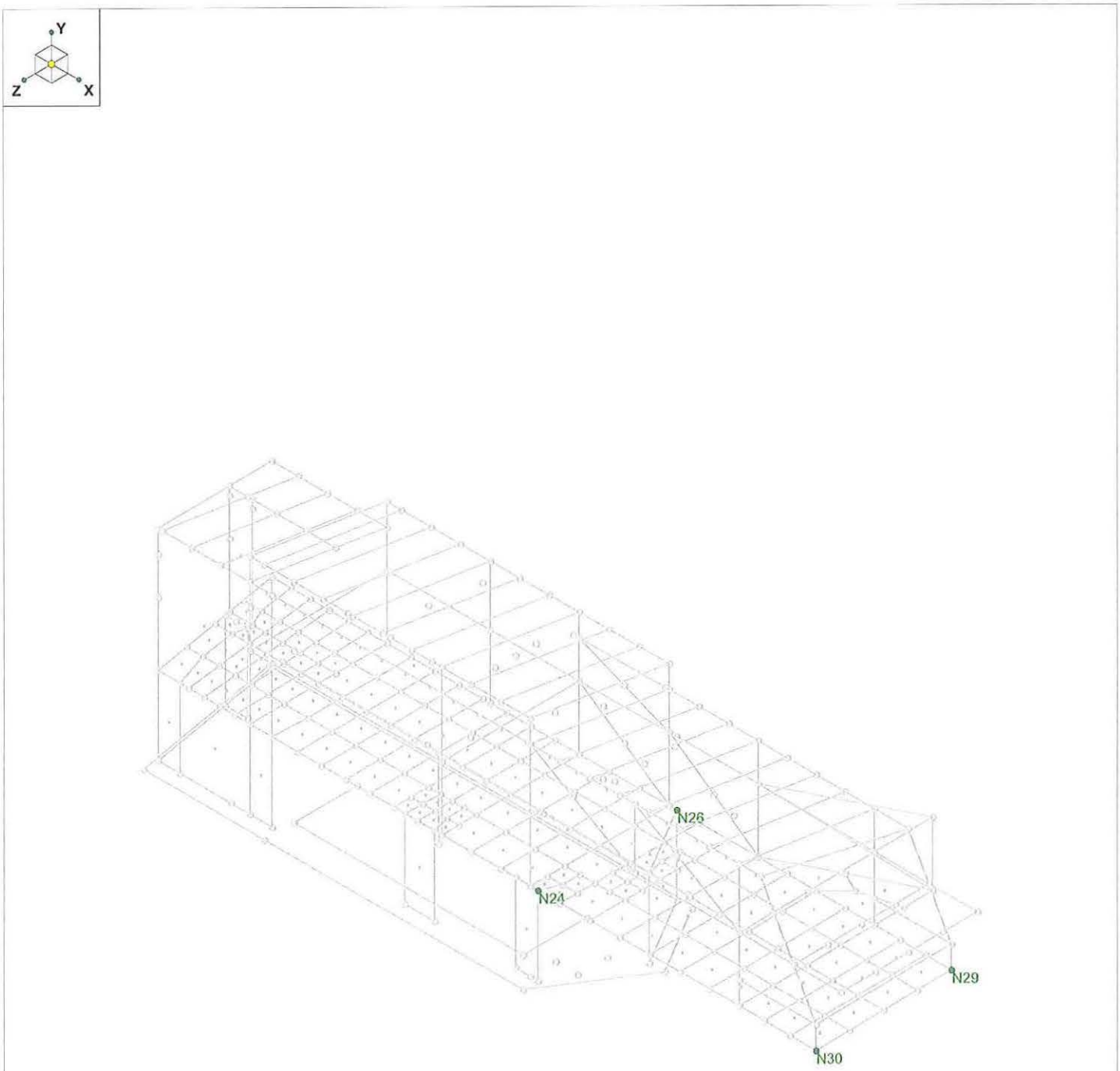
Notes:

1. Code check evaluates demand against code prescribed capacity, and is noted as a ratio. If check is less than 1.Ø (or as permitted by code), then the member is acceptable.
2. Loc: Location along member that controls design.
3. LC: Load combination per ASCE 7-Ø5..
4. V Chk: Shear check expressed as a ratio of demand against code prescribed capacity..
5. Dir: Direction controlling force is applied. Lower case indicates local coordinate sytem.
6. Design Capacities divided by the Omega safety factors: Pnc/Ω, Pnt/Ω, Mnyy/Ω, Mnzz/Ω.
7. Cb: Flexural Bending coefficient per AISC 360-05 Specification.

CODE CHECK, STRENGTH
LOAD COMBINATIONS incl SEISMIC, SKIP UNIFORM LIVE LOADING

Conclusion: Maximum ratio of demand against member capacity = Ø.976, less than 1.Ø.
Members are adequate per DL + LL + E combinations.

Skip loading leads to maximum stress ratios for SEISMIC combinations. Same combinations with full live load do not control.



REFERENCE JOINTS FOR CHECKING DEFLECTION LIMITS

Loads: BLC 16,
Solution: Envelope

DEFLECTION CHECK, UNFACTORED SERVICE LOADS

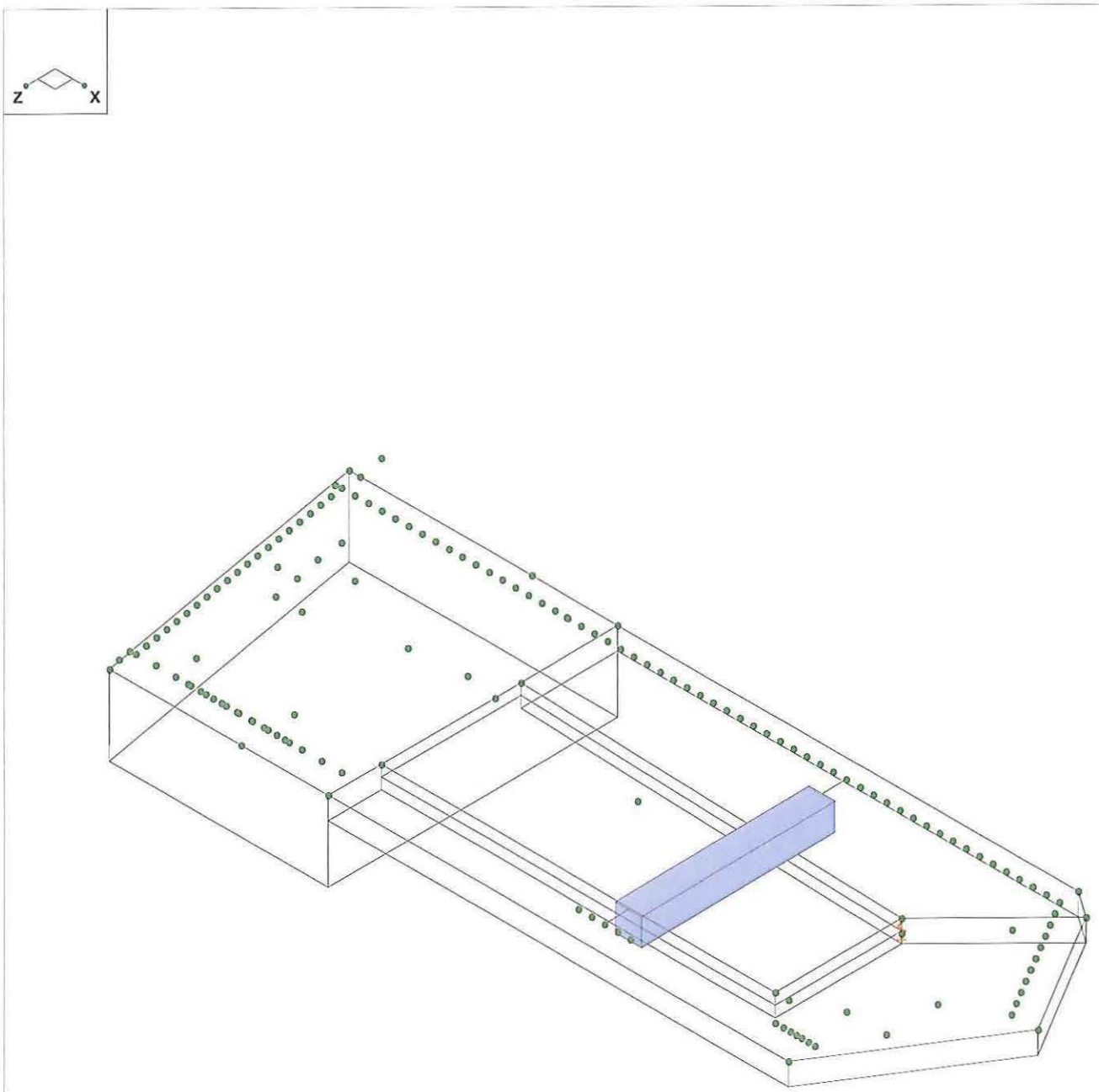
	Joint		Deflection, Inches X	LC	Y	LC	Z	LC	Rotation, Radians X Rotate	LC	Y Rotate	LC	Z Rotate	LC
Uniforcm Live Load	N24	max	0.005	88	-0.018	85	-0.005	87	-0.00002	87	0.00001	85	-0.00050	85
		min	0.004	87	-0.022	88	-0.009	85	-0.00012	86	0.00001	87	-0.00063	88
	N30	max	0.005	88	-0.604	85	-0.009	87	-0.00098	85	0.00001	85	-0.00051	85
		min	0.004	87	-0.739	88	-0.014	85	-0.00128	88	0.00001	87	-0.00060	88
	N26	max	0.003	88	-0.01	85	-0.005	87	0.00002	86	0.00001	85	-0.00044	85
		min	0.002	85	-0.011	88	-0.009	85	0.00000	85	0.00001	87	-0.00053	88
	N29	max	0.003	88	-0.754	85	-0.009	87	0.00026	88	0.00001	85	-0.00063	85
		min	0.002	85	-0.925	88	-0.014	85	0.00015	87	0.00001	87	-0.00074	88
Skipped Live Load	N24	max	0.005	94	-0.018	91	-0.005	93	-0.00002	93	0.00001	91	-0.00050	91
		min	0.004	93	-0.022	94	-0.009	91	-0.00012	92	0.00001	93	-0.00063	94
	N30	max	0.005	94	-0.604	91	-0.009	93	-0.00098	91	0.00001	91	-0.00051	91
		min	0.004	93	-0.739	94	-0.014	91	-0.00128	94	0.00001	93	-0.00060	94
	N26	max	0.003	94	-0.01	91	-0.005	93	0.00002	92	0.00001	91	-0.00044	91
		min	0.002	91	-0.011	94	-0.009	91	0.00000	91	0.00001	93	-0.00053	94
	N29	max	0.003	94	-0.754	91	-0.009	93	0.00026	94	0.00001	91	-0.00063	91
		min	0.002	91	-0.925	94	-0.014	91	0.00015	93	0.00001	93	-0.00074	94

Cantilever Length	37.5 ft
L/360=	1.25 in

	Y (in)	L/Y=
Max Uniform Live Load Deflection	-0.925	486
Max Skipped Live Load Deflection	-0.925	486

Notes::
1. X, Y, Z are expressed relative to the global coordinate system.
2. LC: Load combination.

Conclusion: Maximum service load deflection = 0.925 in is less than L/360 = 1.25 in.
Framing system is adequate for deflection check.



FOUNDATION MODEL, ISOMETRIC
View looking southerly

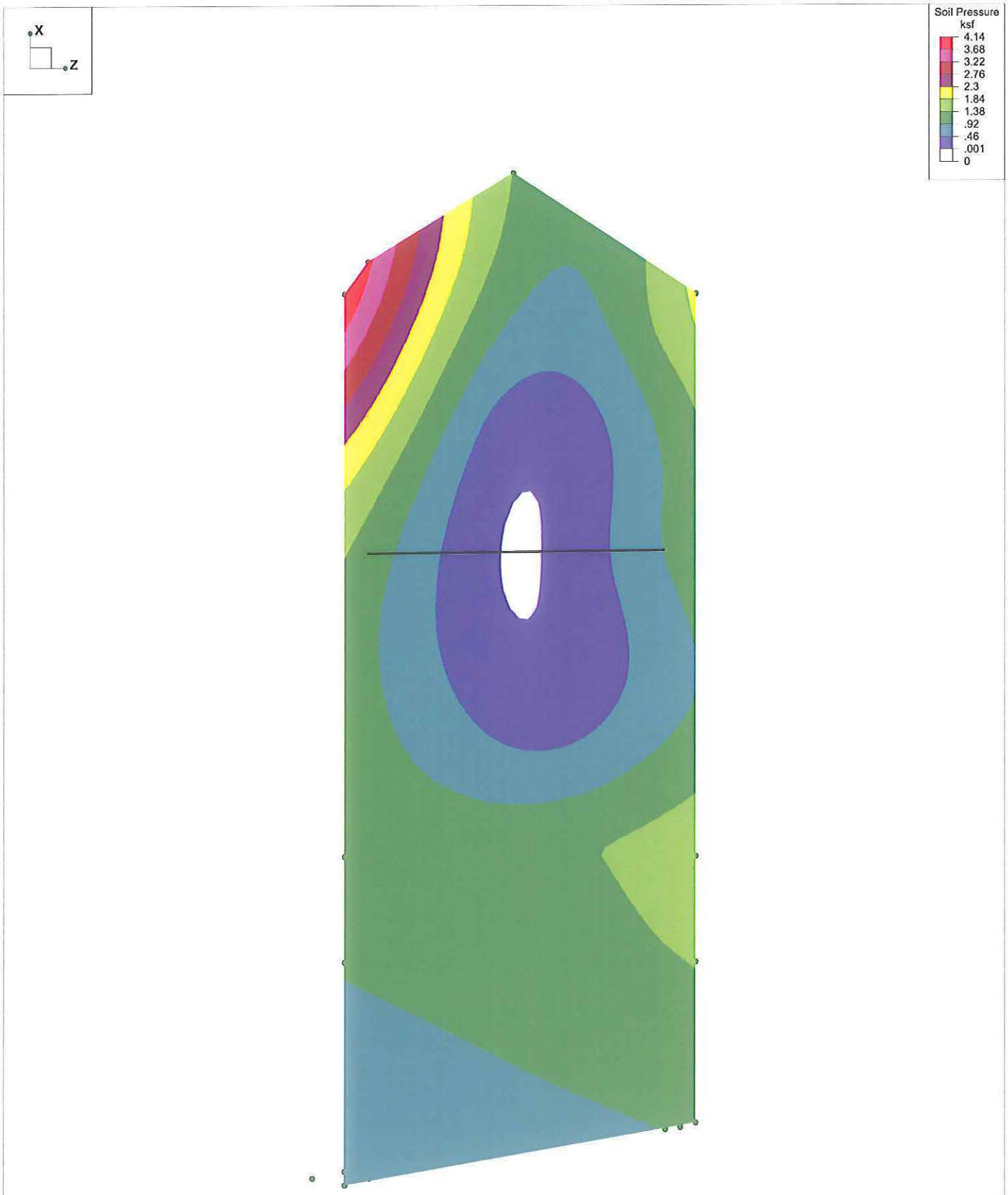
LOAD COMBINATIONS: FOUNDATION
per ASCE 7-05. Refer to Notes for comments

	Description	Solve	Service	ABIF	OSF	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
	Veritcal Comb												
1	ASCE 1	1	1		1.5	DL	1						
2	ASCE 2	1	1		1.5	DL	1	LL	1				
3	ASCE 3 (a)	1	1		1.5	DL	1	RLL	1				
4	ASCE 4 (a)	1	1		1.5	DL	1	LL	0.75	RLL	0.75		
	Eqk Comb												
5	ASCE 5 (b) (a)	1	1	1.33	1.5	DL	1	ELX	0.7				
6	ASCE 5 (b) (b)	1	1	1.33	1.5	DL	1	ELX+Z	0.7				
7	ASCE 5 (b) (c)	1	1	1.33	1.5	DL	1	ELX-Z	0.7				
8	ASCE 5 (b) (d)	1	1	1.33	1.5	DL	1	ELZ	0.7				
9	ASCE 5 (b) (e)	1	1	1.33	1.5	DL	1	ELZ+X	0.7				
10	ASCE 5 (b) (f)	1	1	1.33	1.5	DL	1	ELZ-X	0.7				
11	ASCE 6 (b) (a)	1	1	1.33	1.5	DL	1	ELX	0.525	LL	0.75	RLL	0.75
12	ASCE 6 (b) (b)	1	1	1.33	1.5	DL	1	ELX+Z	0.525	LL	0.75	RLL	0.75
13	ASCE 6 (b) (c)	1	1	1.33	1.5	DL	1	ELX-Z	0.525	LL	0.75	RLL	0.75
14	ASCE 6 (b) (d)	1	1	1.33	1.5	DL	1	ELZ	0.525	LL	0.75	RLL	0.75
15	ASCE 6 (b) (e)	1	1	1.33	1.5	DL	1	ELZ+X	0.525	LL	0.75	RLL	0.75
16	ASCE 6 (b) (f)	1	1	1.33	1.5	DL	1	ELZ-X	0.525	LL	0.75	RLL	0.75
17	ASCE 6 (d) (a)	1	1	1.33	1.5	DL	1	ELX	0.525	LL	0.75		
18	ASCE 6 (d) (b)	1	1	1.33	1.5	DL	1	ELX+Z	0.525	LL	0.75		
19	ASCE 6 (d) (c)	1	1	1.33	1.5	DL	1	ELX-Z	0.525	LL	0.75		
20	ASCE 6 (d) (d)	1	1	1.33	1.5	DL	1	ELZ	0.525	LL	0.75		
21	ASCE 6 (d) (e)	1	1	1.33	1.5	DL	1	ELZ+X	0.525	LL	0.75		
22	ASCE 6 (d) (f)	1	1	1.33	1.5	DL	1	ELZ-X	0.525	LL	0.75		
23	ASCE 8 (a)	1	1	1.33	1.5	DL	0.6	ELX	0.7				
24	ASCE 8 (b)	1	1	1.33	1.5	DL	0.6	ELX+Z	0.7				
25	ASCE 8 (c)	1	1	1.33	1.5	DL	0.6	ELX-Z	0.7				
26	ASCE 8 (d)	1	1	1.33	1.5	DL	0.6	ELZ	0.7				
27	ASCE 8 (e)	1	1	1.33	1.5	DL	0.6	ELZ+X	0.7				
28	ASCE 8 (f)	1	1	1.33	1.5	DL	0.6	ELZ-X	0.7				

LOAD COMBINATIONS: FOUNDATION
per ASCE 7-05. Refer to Notes for comments

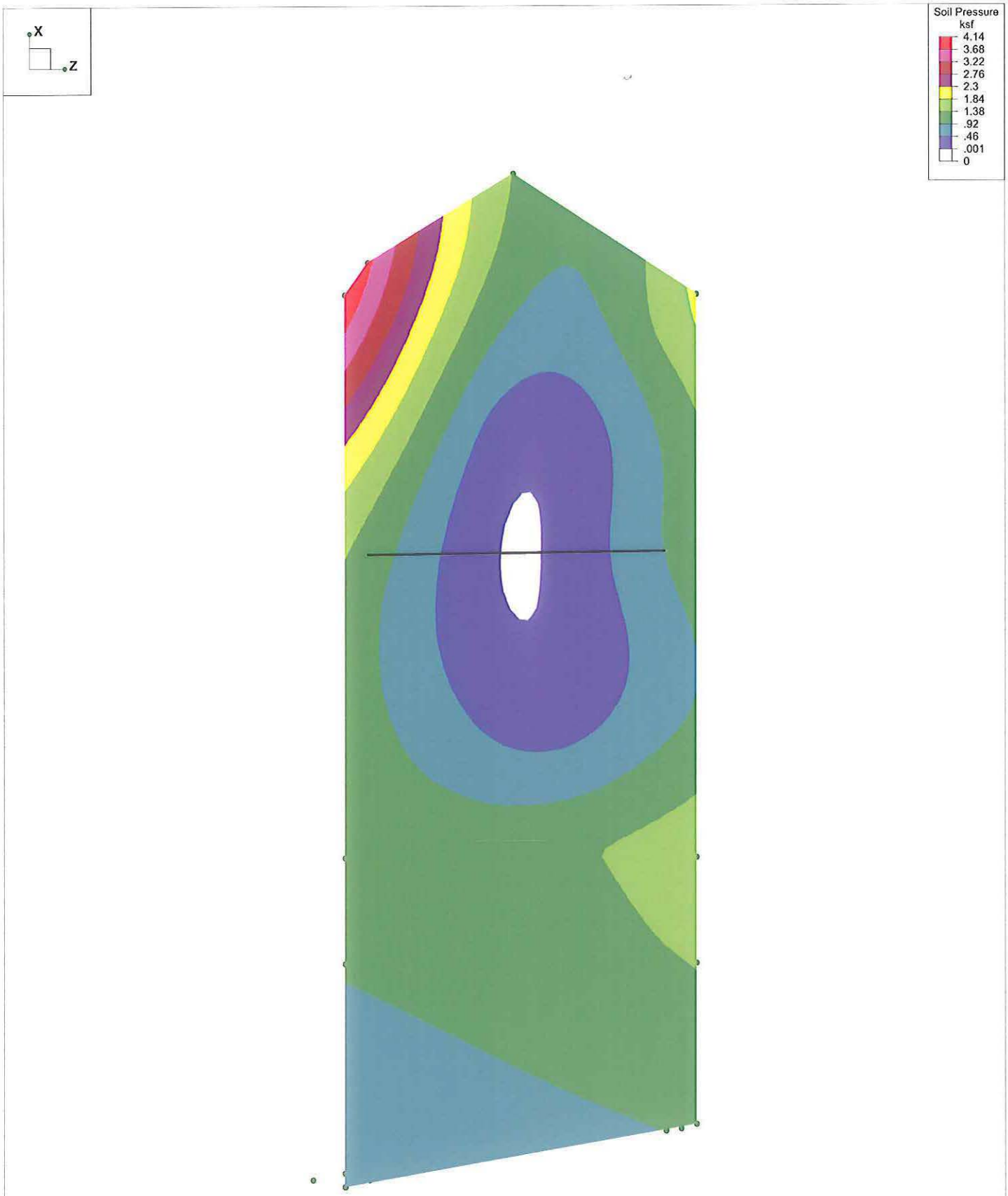
Notes on Foundation Load Combinations:

1. Load combinations are service load combinations used to check soil pressures. Factors applied conform to 7-05 §2.4.1.
2. Load combinations conform to ASCE 7-05 & CBC 2010.
3. ABIF: Allowable bearing increase factor.
4. OSF: Overturning Factor of Safety
5. BLC: Basic Load Case



Results for LC 4, ASCE 4 (a)

Results for LC 7, ASCE 5 (b) (c)



Results for LC 13, ASCE 6 (b) (c)

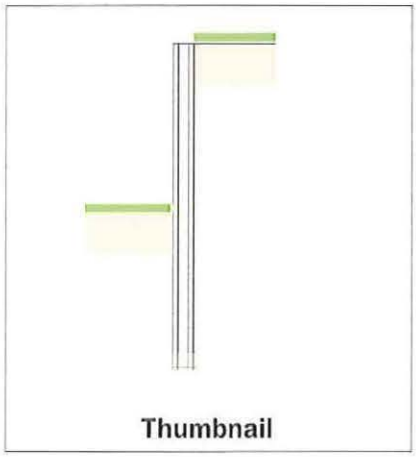
Results for LC 27, ASCE 8 (e)

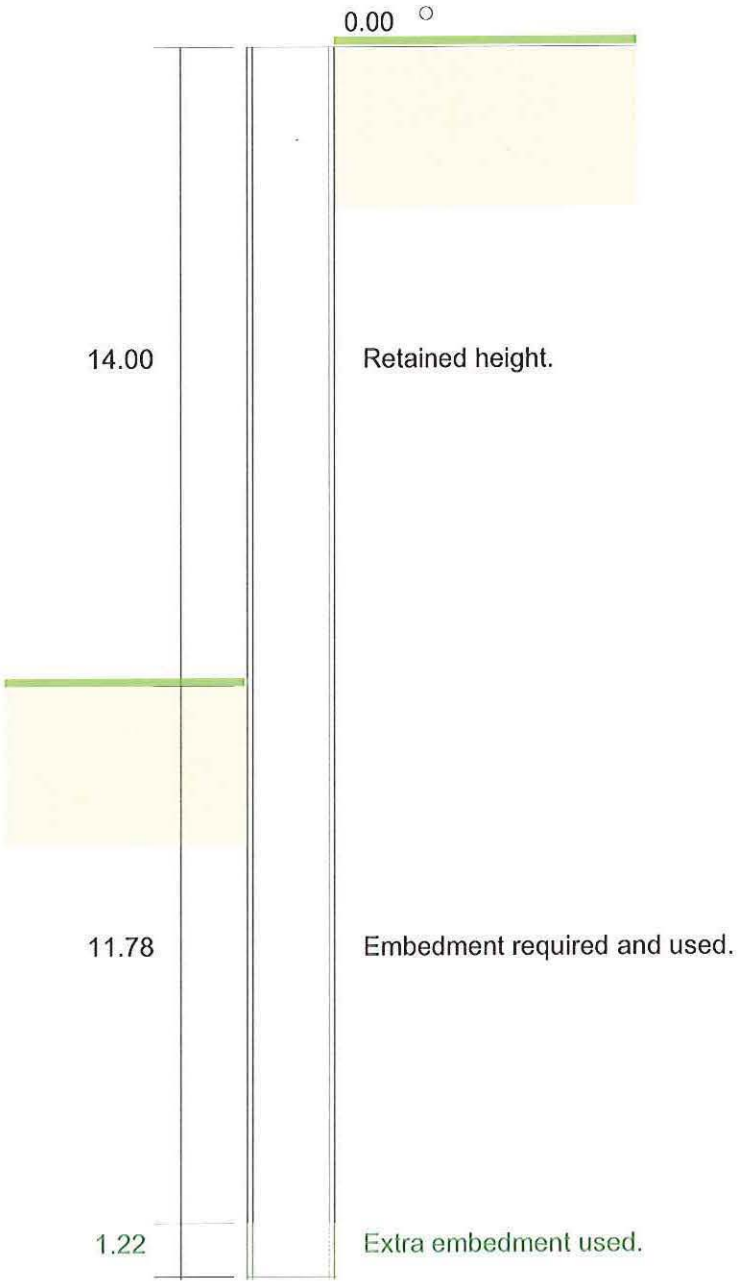
Soldier Pile Retaining Wall Design

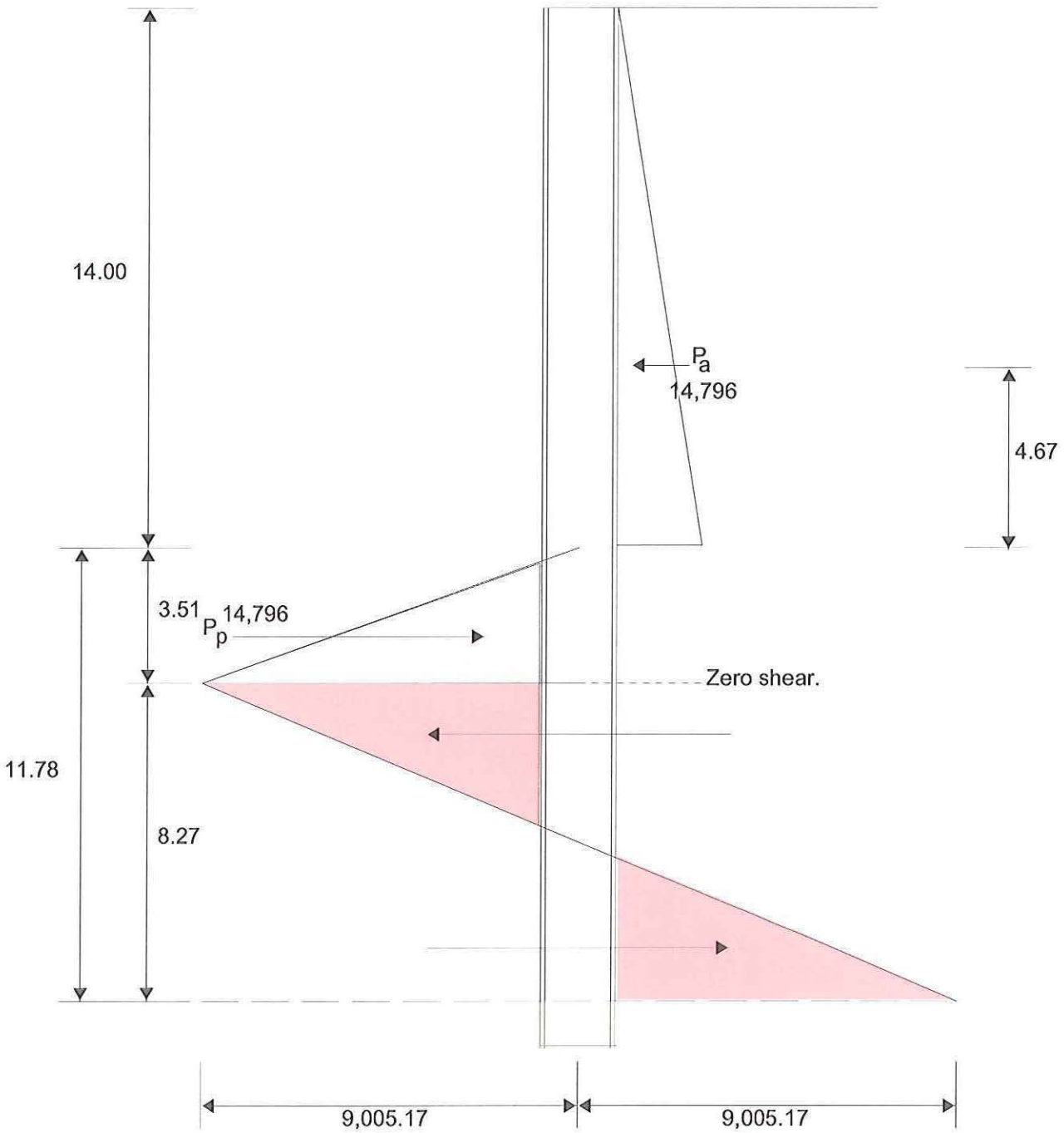
Code: CBC 2010,ACI 318-08,ACI 530-08

Summary

Wall height (retained height), ft	14.00
Backfill slope	0.00
Soil Density, pcf	110.00
Soil Phi angle	45.00
Ka (horizontal)	0.17
Surcharge, psf	0
Allow. Passive, psf / ft. depth	600
Apply S.F. to Passive	1.0
Pile Spacing, ft	8.0
Flange Width, in.	24.00
Multiplier to Passive Wedge	2.0
Required Embedment, ft	11.78
Embedment Used, ft	13.00
Moment in Pile Max, ft-lbs	103,171
Max. Mom. Factored 1.6, LRFD	165,073
Soldier Beam Selection	HP 12x53
Lagging Depth, ft	14.00
Lagging Selection	PL 1




0.00
14.00
Retained height.
11.78
Embedment required and used.
1.22
Extra embedment used.


14.00
Pa
14,796
4.67
3.51
Pp 14,796
Zero shear.
11.78
8.27
9,005.17
9,005.17

UPDATED GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENCE
LOT 41, STUDIO DRIVE
CAYUCOS, CALIFORNIA

December 27, 2011

PROJECT 6-4210

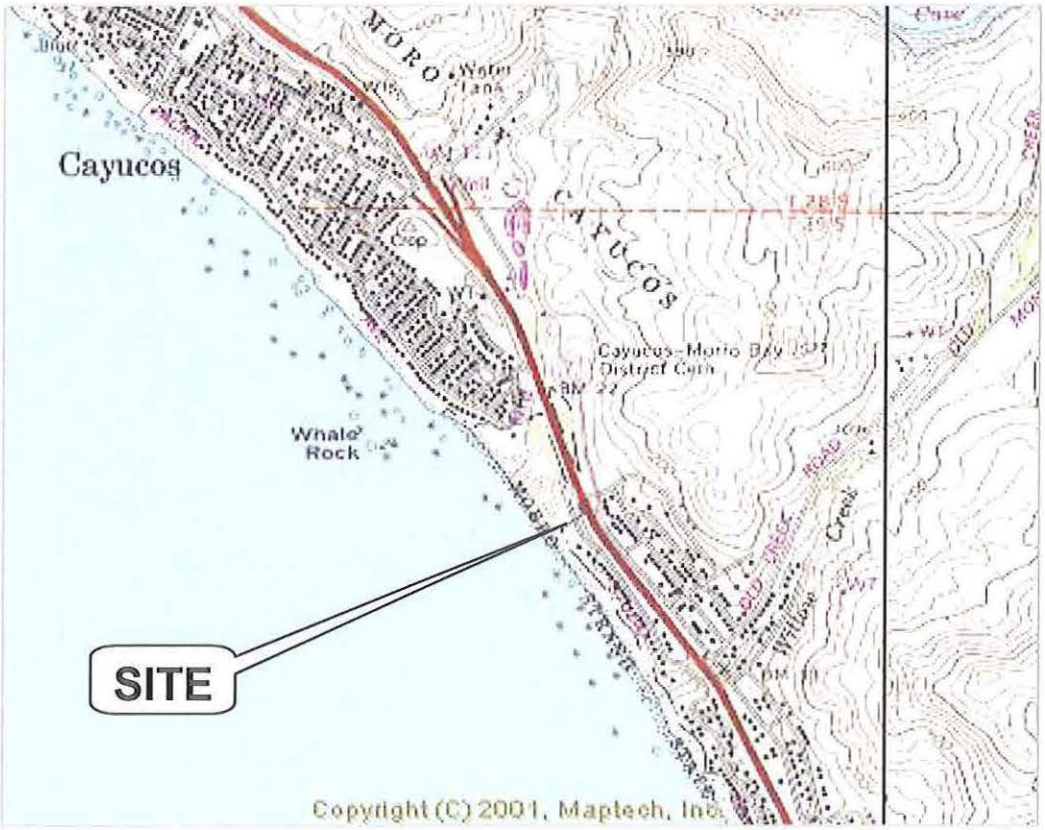


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TABLE OF CONTENTS

1.0 INTRODUCTION 1

2.0 PURPOSE AND SCOPE 2

3.0 SUBSURFACE SOIL CONDITIONS 3

4.0 SEISMIC CONSIDERATIONS 4

5.0 CONCLUSIONS AND RECOMMENDATIONS 5

5.1 Clearing and Stripping 7
5.2 Preparation of Building Pad 7
5.3 Structural Fill 8
5.4 Drilled Piers 9
5.5 Conventional Deepened Foundations 10
5.6 Slab Construction 11
5.7 Retaining Walls 11
5.8 Underground Facilities Construction 13
5.9 Surface and Subsurface Drainage 14
5.10 Temporary Excavations and Slopes 15
5.11 Corrosion 16

6.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS 17

FIGURES
Site Location Map
Site Plan
Soldier Pile Shoring
Shoring Schematic
Liquefaction Analysis

APPENDIX A
Soil Classification Chart
Logs of Exploratory Borings and Trenches

APPENDIX B
Moisture-Density Tests
Direct Shear Test
Expansion Index Test
Consolidation Tests

UPDATED GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENCE
LOT 41, STUDIO DRIVE
CAYUCOS, CALIFORNIA

PROJECT 6-4210

1.0 INTRODUCTION

This report presents the results of our updated geotechnical investigation for the proposed residence to be located on Lot 41, Studio Drive in Cayucos, California (APN: 064-253-070). A site location map is presented in Figure 1. A geotechnical investigation was previously performed for the residence by GSI Soils Inc (GSI) in 2007 (project No 6-4120, dated January 12, 2007). The propose of this updated report is to address the questions raised in the peer review (by Cotton & Shires, dated May 31, 2011) of the 2007 GSI report and to provide recommendations for installation of a shoring system to support the adjacent property during basement excavation.

The property is bounded by Studio Drive to the east, a residence to the south and Morro Strand State Beach to the north and west. In general the site slopes to the north and west with an average elevation of approximately 20 feet above mean sea level (MSL). The west side of the lot is an active beach and is relatively level. The east side of the site is undulating and covered with iceplant. Bedrock outcrops were also observed adjacent to the beach. A concrete lined drainage swale is located adjacent to Studio Drive.

It is our understanding that the residence will be constructed on a structural mat slab supported on deepened/deadman footings and/or drilled piers. Based on discussions with Bruce Elster of Shoreline Engineering the deepened/deadman footing on the east side of the residence will extend the full width of the residence, be approximately 7 to 8 feet deep and 18 feet long. The purpose of the deadman footings will be to resist the cantilever loading of the westside of the residence which extends some 28 feet over the active beach area The mat slab will be located at basement level at an elevation of approximately 15 feet above MSL. Cuts varying from approximately 5 feet on the north side of the pad to 12 feet below existing grades on the south side are anticipated. To retain the cuts a system of soldier beams and steel plate lagging will be used. Retaining walls will also be constructed adjacent to Studio Drive with continuous footings extending into the underlying bedrock materials. Footing loads for the building and retaining walls are presently unavailable. For the purpose of this report, deepened footings loads on the

order of 5 kips per lineal foot, and pier loads of approximately 200 kips are anticipated for the residence and 3.0 kips per lineal foot for the retaining walls to be located adjacent to Studio Drive.

The project description is based on a site reconnaissance performed by a GSI Soils Inc., engineer and information provided by Shoreline Engineering. The plan provided (by Volbrecht Surveys) forms the basis for the "Site Plan", Figure 2.

In the event that there is change in the nature, design or location of improvements, or if the assumed loads are not consistent with actual design loads, the conclusions and recommendations contained in this report should be reviewed and modified, if required. Evaluations of the soils for hydrocarbons or other chemical properties are beyond the scope of the investigation.

2.0 PURPOSE AND SCOPE

The purpose of this study was to further explore and evaluate the surface and subsurface soil conditions at the site and to develop updated geotechnical information and design criteria for the
proposed project. The scope of this study included the following items.

1. A review of available soils information for this area of Cayucos.

2. A field study consisting of a site reconnaissance and additional exploratory borings to further evaluate the subsurface conditions.

3. A laboratory testing program performed on representative soil samples collected during our field study.

4. Engineering analysis of the data gathered during our recent and previous field studies, laboratory testing, and literature review. Development of recommendations for site preparation and grading, and updated geotechnical design criteria for foundations, temporary shoring, retaining walls, and underground facilities.

5. Preparation of this report summarizing our findings, conclusions, and recommendations regarding the geotechnical aspects of the project site.

3.0 SUBSURFACE SOIL CONDITIONS

The near surface soils encountered in our exploratory borings and trenches generally consisted of undocumented fill materials with debris to a depth of 4.5 to 6 feet. These materials consisted of sandy clays, clayey sands and sands and were encountered in a soft and/or loose condition. The debris generally consisted of concrete and asphalt pieces in the upper 3 to 5 feet. Plastics and bottles were also observed below a depth of 1 to 2 feet in boring B-2 and in both trenches. Below the fill materials in borings B-1 and B-2, clayey sands and sandy clays were encountered over hard bedrock (Graywacke Sandstone). These materials were found in a moist state and in a dense to very dense condition. The bedrock became very hard and the auger met with refusal at 7 to 9 feet below grade. In boring B-3 sands were encountered below the fill materials to a depth of 12.5 feet. These soils were found in a moist state and in a loose condition. Below these sands, very dense clayey sands were encountered over hard bedrock. In all of the borings refusal to auger penetration was encountered in the graywacke sandstone. The depth to refusal varied from approximately 7 to 14 feet below existing grades.

Two additional borings (B-4 & B-5) were drilled at the site adjacent to the existing residence to the south (2612 Studio Drive). In both of the borings 4 to 5 feet of sandy clay fill materials were encountered in a firm to stiff condition. Native sandy clays in a very stiff condition were found below the fills to a depth of 6 to 8 feet. Very dense clayey sandstone was encountered underlying the clays. Refusal in these materials occurred at 8 to 9.5 feet. A sample of the hard sandstone material was tested for compressive strength. The results of this test indicate that the materials have a compressive strength of 1240 psi.

Free ground water was not encountered during our field exploration. However, perched water conditions in the upper 5 feet should be anticipated during wet winter months above the dense bedrock materials. A more detailed description of the soils encountered is presented graphically on the "Exploratory Boring and Trench Logs", B-1 through B-5, T-1 and T-2, Appendix A. An explanation of the symbols and descriptions used on the logs are presented on the "Soil Classification Chart". Section views are also provided on Figure 2.

The soil profile described above is generalized; therefore, the reader is advised to consult the boring and trench logs (Appendix A) for soil conditions at specific locations. Care should be exercised in interpolating or extrapolating subsurface conditions between and beyond the borings and trenches. On the boring and trench logs we have indicated the soil type, moisture content, grain size, dry density, and the applicable United Soil Classification System Symbol.

The locations of our exploratory and trench borings, shown on Site Plan, Figure 2, were approximately determined from features at the site. Hence, accuracy can be implied only to the degree that this method warrants. The surface elevation at the boring and trench locations was not determined.

4.0 SEISMIC CONSIDERATIONS

4.1 Seismic Coefficients

The project site was positioned on the USGS Seismic Hazard Maps for a 2% probability of exceedance in 50 years to determine the maximum considered earthquake spectral response accelerations. The design acceleration coefficients for short periods (S_{DS}) and at 1-second (S_{D1}) were found to be 0.980g and 0.491g respectively. A site class C should be used of design of structures.

4.2 Seismic Settlements

Liquefaction is described as the sudden loss of soil shear strength due to a rapid increase of pore water pressures caused by cyclic loading from a seismic event. In simple terms it means that the soil acts more like a fluid than a solid in a liquefiable event. In order for liquefaction to occur, the following are generally needed; granular soils (sand, silty sand and sandy silt), groundwater and low density (very loose to medium dense) conditions. Undocumented fill materials (sandy clays) were found to a depth of 4.5 to 6 feet across the building pad area. Very stiff sandy clays and bedrock materials were generally encountered below these fill materials. However, in the middle of the building pad area, loose sands were found to a depth of 12.5 feet. Below the sands weathered bedrock materials were encountered over hard bedrock. Groundwater was not found in the borings. For the purposes of this study the sands encountered from 6 to 12.5 feet was assumed to be saturated.

To evaluate the potential for liquefaction, the soil and seismic parameters were inputted and an analysis performed utilizing the software program Liquefypro by Civiltech. A peak ground acceleration (PGA) of 0.60g based on a magnitude 7.2 earthquake on the Hosgri Fault was used in the program. The SPT blow counts obtained during our field exploration were converted to $N_1(60)$ values using this program. The program also determines the potential settlements and Factors of Safety (FOS). The results of this analysis are summarized on the attached Figure 5. Overall (total) seismic settlements on the order of 1-3/4 inches were obtained with Factors of Safety (FOS) of less than 1.0 for the sand soils encountered from 6 to 12 feet below grade. Differential settlements are typically 60 percent of the total settlements or approximately 1-inch. A further check of liquefaction induced settlements was determined based on the grading recommendations provided in Section 5.2 of this report. This grading would result in 3 feet of compacted decomposed granite over at least 1 foot of compacted native sands. This analysis indicates that total settlements will be on the order of ½-inch (see Figure 6). These seismic settlements are relatively minor and are not expected to influence the proposed foundation system.

Due to the limited depth of sand (approximately 6 feet) within the building pad area, dry settlements of these sands during seismic ground shaking is expected to be less than ½ inch. In addition, with the proposed grading, these settlements are anticipated to be less than ¼ inch.

4.3 Lateral Spreading

Considering the proposed foundation system, site grading and the confined condition of the sands near the center of the pad, the potential for lateral spreading displacements would be negligible.

5.0 CONCLUSIONS AND RECOMMENDATIONS

1. The site is suitable for the proposed residence provided the recommendations presented in this report are incorporated into the project plans and specifications.

2. The primary geotechnical concerns are the large uplift loads, the undocumented fills across the east side of the site and the cuts required adjacent to the

residence to the south. Deepened/deadman foundations extending on the order of 4 to 7 feet below existing grades and/or drilled piers extending a minimum of 15 feet into the graywacke sandstone bedrock are being considered to support the residence. In addition, a structural mat slab will be used to tie the foundation system to the deepened footings and/or drilled piers. To retain the cuts required to construct the basement level of the residence, soldier beams and steel plate lagging will be used. This shoring system will provide continuous support of the cuts while the site materials are excavated.

3. Due to the presence of relatively shallow bedrock materials and the absence of groundwater the potential for liquefaction would, in our opinion, be in the low to negligible category.

4. All grading and foundation plans should be reviewed by GSI Soils Inc., hereinafter described as the Geotechnical Engineer, prior to contract bidding. This review should be performed to determine whether the recommendations contained within this report are incorporated into the project plans and specifications.

5. The Geotechnical Engineer should be notified at least two (2) working days before site clearing or grading operations commence, and should be present to observe the stripping of deleterious material and provide consultation to the Grading Contractor in the field.

6. Field observation and testing during the grading operations should be provided by the Geotechnical Engineer so that a decision can be formed regarding the adequacy of the site preparation, the acceptability of fill materials, and the extent to which the earthwork construction and the degree of compaction comply with the project geotechnical specifications. Any work related to grading performed without the full knowledge of, and under direct observation of the Geotechnical Engineer, may render the recommendations of this report invalid.

5.1 Clearing and Stripping

1. All surface and subsurface deleterious materials should be removed from the proposed building area and disposed of off-site. This includes, but is not limited to any buried utility lines, loose fills, debris, building materials, and any other surface and subsurface structures within proposed building areas. Voids left from site clearing, should be cleaned and backfilled as recommended for structural fill. Once the site has been cleared, the exposed ground surface should be stripped to remove surface vegetation and organic soil.

5.2 Preparation of Building Pad

1. As discussed previously, the residence will be supported on deepened/deadman footings and/or drilled piers. It is also anticipated that the driveway will span between the residence and Studio Drive. A retaining wall will likely be required adjacent to Studio Drive. The footings for this driveway should extend a minimum of 12 inches into the underlying graywacke sandstone. If compacted select soils are to be placed for the driveway all existing fills should be removed to competent materials.

2. As indicated above, a shoring system will be constructed to retain the proposed cuts to establish the basement grade. After installation of the shoring system, the building pad area should be excavated to a minimum depth of 3 feet below the bottom of the proposed mat slab. In addition, all fill materials and debris should be removed to competent native soils. Where native sands are exposed a layer of geofabric (Mirafi HP570 or equivalent) should be placed across the surface. A minimum of 3 feet of select fill materials (decomposed granite or Class II Base) should be placed and compacted to 90 percent of ASTM D1557-02 to pad grade. This grading combined with the proposed foundation system will eliminate the potential effects of expansive soils at the site.

3. In order to help minimize potential settlement problems the soils engineer should be consulted for specific site recommendations during construction.

4. The above recommendations are based on the strength characteristics of the materials under conditions of normal moisture that would result from rain water

and do not take into consideration the additional activating forces applied by seepage from springs or subsurface water. Areas of observed seepage should be provided with subsurface drains to release the hydrostatic pressures.

5. The near-surface soils may become partially or completely saturated during the rainy season. Site grading operations during this time period may be difficult since the saturated materials may not be compactable and they may not support construction equipment. Consideration should be given to the seasonal limit of the grading operations on the site.

6. All final grades should be provided with a positive drainage gradient away from foundations. Final grades should provide for rapid removal of surface water runoff. Ponding of water should not be allowed on building pads or adjacent to foundations.

5.3 Structural Fill

1. On-site sand soils free of organic and deleterious material are suitable for use as non-structural fill below the select fill cap. These fills should not contain rocks larger than 3 inches in greatest dimension, and should have no more than 15 percent larger than 1.5 inches in greatest dimension.

2. Select import (decomposed granite or Class II/III Base) should be free of organic and other deleterious material and should have a very low expansion potential with a plasticity index of 10 or less. Before delivery to the site, a sample of the proposed import should be tested in our laboratory to determine its suitability for use as structural fill.

3. Structural fill using on-site inorganic soil or approved import should be placed in layers, each not exceeding eight inches in thickness before compaction. On-site inorganic or imported soil should be conditioned with water, or allowed to dry, to produce a soil water content at approximately optimum value, and should be compacted to at least 90 percent relative compaction based on ASTM D1557-02.

5.4 Drilled Piers

1. As indicated previously, the residence may be supported on drilled piers. The piers should be embedded into the underlying bedrock materials a minimum of 15 feet. Due to the variability in the surface fill materials, piers are likely to extend to depths on the order of 25 to 30 feet below existing grades. To accommodate the variability of the materials and to adjust the pier lengths accordingly, a representative of GSI Soils should be continuously on-site to monitor drilling, rebar placement and concrete placement.

2. Piers should have a minimum diameter of 18 inches and be designed using skin friction. An allowable skin friction value of 2000 psf may be used for downward loads in the competent greywacke sandstone bedrock. A value of 1600 psf may be used for uplift loads in the same material.

3. A lateral bearing value of 400 pcf (for graywacke bedrock) together with the methods described in the California Building Code can be used for lateral capacity calculations.

4. The soils generally consisted of clay soils over bedrock materials. Casing will likely not be required in these materials to install the piers. Where sands are found a shallow casing may be required to prevent caving and sloughing into the hole.

5. The bedrock materials encountered were in a very dense to hard condition and special equipment (rock core bit) will be required to reach design depths.

6. Concrete should have a maximum water to cement ratio of 0.45 and have a minimum compressive strength of 4,000 psi to reduce the potential for seawater attack. It would also be prudent to use air entrainment and a corrosion inhibitor in the concrete. Epoxy coating of the reinforcing steel could also be considered to protect the reinforcing steel. For concrete placed in dry excavations a 4 to 6-inch slump would be appropriate. For concrete placed by the tremie method a 7 to 9-inch slump is recommended. The shaft(s) should contain steel reinforcement as determined by the Project Structural Engineer in accordance

with applicable CBC or ACI Standards.

7. The grade beam/foundations connecting the drilled piers should extend into the underlying bedrock at least 12 inches or into competent materials approved by the geotechnical engineer with a minimum overall depth of four (4) feet below basement grade.

5.5 Conventional Deepened Foundation

1. Conventional deepened continuous concrete footings that will also act as deadman footings to counteract the cantilever loads can also be used to support the residence. The footing excavations will extend on the order of 7 to 8 feet below grade on the east side of the residence and on the order of 4 feet on the west side. However, in all cases the footings should extend at least 12 inches into the underlying greywacke sandstone. The exposed bearing surfaces should then be evaluated by the geotechnical engineer.

2. For footings a minimum of 2 feet wide and extending a minimum of 12 inches into the underlying bedrock and 4 feet below lowest adjacent grade an allowable dead plus live load bearing pressure of 3500 psf may be used. For each addition one-foot of footing width and depth the bearing pressure can be increased by 200 psf and 500 psf respectively up to a maximum allowable bearing pressure of 5000 psf

3. Allowable bearing capacities may be increased by one-third (⅓) when transient loads such as winds or seismicity are included. Structural settlements of less than 1.0 inch are anticipated with differential settlement on the order of 50 percent of these values. Reinforcement for the footings should be designed by the structural engineer.

4. Lateral forces on structures may be resisted by passive pressure acting against the sides of shallow footings and/or friction between the soil and the bottom of the footing. For resistance to lateral loads, a friction factor of 0.40 may be utilized for sliding resistance at the base of the spread footings in undisturbed native bedrock materials. A passive resistance of 450 pcf equivalent fluid weight

may be used against the side of shallow footings. If friction and passive pressures are combined, the lesser value should be reduced by 33 percent

5.6 Slab Construction

1. Concrete slabs and flatwork should not be placed directly on unprepared loose fill materials. Preparation of subgrade to receive concrete slabs-on-grade and flatwork should be processed as discussed in the preceding sections of this report.

2. To minimize floor dampness a section of capillary break material at least 4 inches thick and covered with a 10-mil polyethylene barrier should be provided between slabs-on-grade and compacted soil subgrade. The capillary break should be a clean free-draining material such as clean gravel or permeable aggregate complying with Caltrans Standard Specifications 68, Class I, Type A or Type B, to service as a cushion and a capillary break. Clean gravel should have less than 3% passing the No. 200 sieve. All seams through the vapor barrier should be overlapped and sealed. Where pipes extend through the vapor barrier, the barrier should be sealed to the pipes. Tears or punctures in the moisture barrier should be completely repaired. It is suggested that a 2 inch thick sand layer be placed on top of the membrane to assist in the curing of the concrete. The sand should be lightly moistened prior to placing concrete.

3. Concrete slabs should be a minimum of 8 inches thick and should be reinforced with at least two mats of No. 4 reinforcing bars placed at 18 inches on-center both ways. The aforementioned slab thickness and reinforcement may be used for uniform floor loads not exceeding 200 psf.

4. For design of concrete floors, a modulus of subgrade reaction of k = 150 pci would be applicable to on-site engineered fill soils.

5. 7 Retaining Walls

1. Retaining walls should be designed to resist lateral pressures from adjacent soils and surcharge loads applied behind the walls.

Lateral Pressure and Condition (Compacted Fill)		Equivalent Fluid Pressure, pcf	
		Unrestrained Wall	Rigidly Supported Wall
Active Case, Drained	Level-native soils	60	--
	Level-granular backfill	30	--
At-Rest Case, Drained	Level-native soils	--	75
	Level-sand backfill		50
Passive Case, Drained	Level 2:1 Sloping Down	300 125	--
For sloping backfill add 1 pcf for every 2 deg. (Active case) and 1.5 pcf for every 2 deg. (At-rest case)			

2. Retaining wall foundations should extend a minimum depth of 12 inches into the underlying graywacke sandstone bedrock materials with a minimum overall depth of 36 inches below lowest adjacent grade. An allowable toe pressure of 3,500 psf is recommended in competent bedrock. A coefficient of friction of 0.35 may be used between bedrock and concrete footings.

3. For retaining walls greater than 12 feet, as measured from the top of the foundation, a seismic horizontal surcharge of $10H^2$ (pounds per linear foot of wall) may be assumed to act on retaining walls. The surcharge will act at a height of 0.6H above the wall base (where H is the height of the wall in feet). This surcharge force shall be added to an active design equivalent fluid pressure of 45 pounds per square foot of depth for the seismic condition.

4. In addition to the lateral soil pressure given above, retaining walls should be designed to support any design live load, such as from vehicle and construction surcharges, etc., to be supported by the wall backfill. If construction vehicles are required to operate within 10 feet of a wall, supplemental pressures will be induced and should be taken into account through design.

5. The above-recommended pressures are based on the assumption that sufficient subsurface drainage will be provided behind the walls to prevent the build-up of hydrostatic pressure. To achieve this we recommend that a filter material be

placed behind all proposed walls. The blanket of filter material should be a minimum of 12 inches thick and should extend from the bottom of the wall to within 12 inches of the ground surface. The top 12 inches should consist of water conditioned, compacted native soil. A 4-inch diameter drain pipe should be installed near the bottom of the filter blanket with perforations facing down. The drain pipe should be underlain by at least 4 inches of filter type material. Adequate gradients should be provided to discharge water that collects behind the retaining wall to an adequately controlled discharge system with suitably projected outlets. The filter material should conform to Class I, Type B permeable material as specified in Section 68 of the California Department of Transportation Standard Specifications, current edition. A typical 1" x #4 concrete coarse aggregate mix approximates this specification.

6. For hydrostatic loading conditions (i.e. no free drainage behind walls), an additional loading of 45 pcf equivalent fluid weight should be added to the above soil pressures. If it is necessary to design retaining structures for submerged conditions, allowed bearing and passive pressures should be reduced by 50 percent. In addition, soil friction beneath the base of the foundations should be neglected.

7. Precautions should be taken to ensure that heavy compaction equipment is not used immediately adjacent to walls, so as to prevent undue pressure against, and movement of, the walls.

5.8 Underground Facilities Construction

1. The attention of contractors, particularly the underground contractors, should be drawn to the State of California Construction Safety Orders for "Excavations, Trenches, Earthwork". Trenches or excavations greater than 5 feet in depth should be shored or sloped back in accordance with OSHA Regulations prior to entry.

2. For purposes of this section of the report, bedding is defined as material placed in a trench up to 1 foot above a utility pipe and backfill is all material placed in the trench above the bedding. Unless concrete bedding is required around utility

pipes, free-draining sand should be used as bedding. Sand proposed for use as bedding should be tested in our laboratory to verify its suitability and to measure its compaction characteristics. Sand bedding should be compacted by mechanical means to achieve at least 90 percent relative compaction based on ASTM Test D1557-02.

3. On-site inorganic soil, or approved import, may be used as utility trench backfill. Proper compaction of trench backfill will be necessary under and adjacent to structural fill, building foundations, concrete slabs and vehicle pavements. In these areas, backfill should be conditioned with water (or allowed to dry), to produce a soil water content of about 2 to 3 percent above the optimum value and placed in horizontal layers each not exceeding 8 inches in thickness before compaction. Each layer should be compacted to at least 90 percent relative compaction based on ASTM Test D1557-02. The top lift of trench backfill under vehicle pavements should be compacted to the requirements given in report section 5.3 for vehicle pavement subgrades. Trench walls must be kept moist prior to and during backfill placement.

5.9 Surface and Subsurface Drainage

1. Concentrated surface water runoff within or immediately adjacent to the site should be conveyed in pipes or in lined channels to discharge areas that are relatively level or that are adequately protected against erosion.

2. Water from roof downspouts should be conveyed in pipes that discharge in areas a safe distance away from structures. Surface drainage gradients should be planned to prevent ponding and promote drainage of surface water away from building foundations, edges of pavements and sidewalks. For soil areas we recommend that a minimum of two (2) percent gradient be maintained with an increase to four (4) percent for the first five (5) feet adjacent to the structure.

3. Careful attention should be paid to erosion protection of soil surfaces adjacent to the edges of roads, curbs and sidewalks, and in other areas where "hard" edges of structures may cause concentrated flow of surface water runoff. Erosion

resistant matting such as Miramat, or other similar products, may be considered for lining drainage channels.

4. Subdrains should be placed in established drainage courses and potential seepage areas. The location of subdrains should be determined during grading. The subdrain outlet should extend into a suitable protected area or could be connected to the proposed storm drain system. The outlet pipe should consist of an unperforated pipe the same diameter as the perforated pipe.

5.10 Temporary Excavations and Slopes

1. Conventional earth moving equipment should be adequate to excavate the soils at the site. As indicated previously, to excavate the hard bedrock materials special equipment (rock bits) maybe required.

2. The proposed cuts adjacent to the residence to the south and to establish basement grade will be retained with soldier piles and steel plate lagging. It should be noted that it is the *Contractor's* responsibility to maintain safe cut slopes based on actual field conditions and according to OSHA requirements. In no case should personnel enter trenches with vertical sidewalls greater than 5 feet deep without proper shoring. Design and installation of the shoring should be the responsibility of the *Contractor* and should be performed according to OSHA requirements.

3. Shoring should be designed to resist the lateral earth pressures assuming no hydrostatic loads. If ground water is encountered the shoring should be designed for the required hydrostatic pressures. The design and maintenance of shoring is the responsibility of the contractor.

4. The lateral pressures recommended for design of the shoring system (soldier piles and lagging) are presented on Figure 4. Lagging between soldier piles can be designed using two-thirds of these pressures.

5. A surcharge load due to equipment within about 10 feet of the back of the shoring should be added to the lateral earth pressures presented above. The

surcharge load can be determined using a uniform pressure of 100 psf over the upper 10 feet of the shoring.

6. The lateral (passive) resistance for cantilevered shoring should be determined using an equivalent fluid weight of 400 pcf. For soldier piles spaced at least two pile-widths apart, the passive pressure can be assumed to act on an area of two pile widths.

7. The soldier piles may be installed by drilling and grouting.

8. Penetration of the soldier piles must be sufficient to achieve lateral stability and resist the downward loading of tiebacks. Vertical loads can be resisted by a combination of 1) skin friction between the back of the soldier pile and the adjacent soil above the excavation bottom, and 2) skin friction on all sides of the soldier pile below the excavation. Above the excavation bottom a skin friction value of 500 psf may be used. Skin friction of 2000 psf may be used below the bottom of the excavation. End bearing should be neglected. Soldier piles should be embedded a minimum of 10 feet below the bottom of the excavation.

9. We recommend that survey points be established on the adjacent property to monitor any deflection.

5.11 Corrosion

1. To provide corrosion control guidelines, soil samples were obtained for resistivity testing. Testing was performed on a samples obtained from test boring B-2. The results are presented on the following table.

SOIL RESISTIVITY	
Sample Location	Soil Resistivity (ohm-cm)
B-2 @ 2 feet	7,100
B-2 @ 6 feet	2,800

2. Two (2) soil samples were tested to measure pH, and the concentration of sulfates and chlorides. The results are presented in the following table.

CHEMICAL TESTS					
Sample Location	Depth	Soil Type	Sulfates (ppm)	Chlorides (ppm)	pH
B-2	2.0'	Silty Sand	4	14	8.0
B-2	6.0'	Sandy Clay	3	11	8.3

3. The results indicate that sulfate salt content is in the negligible range and should not effect normally formulated concrete (Type II Cement). We would recommend using a minimum 4,000 psi concrete with a maximum water/cement ratio of 0.45. The resistivity measurements indicate that the potential for corrosion of ferrous pipes is in the mild corrosive range.

6.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. It should be noted that it is the responsibility of the owner or his/her representative to notify **GSI Soils Inc.** a minimum of 48 hours before any stripping, grading, or foundation excavations can commence at this site.

2. The recommendations of this report are based upon the assumption that the soil conditions do not deviate from those disclosed during our study. Should any variations or undesirable conditions be encountered during grading of the site, **GSI Soils Inc.** will provide supplemental recommendations as dictated by the field conditions.

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

4. As of the present date, the findings of this report are valid for the property studied. With the passage of time, changes in the conditions of a property can occur whether they be due to natural processes or to the works of man on this or adjacent properties. Legislation or the broadening of knowledge may result in changes in applicable standards. Changes outside of our control may find this report to be invalid, wholly or partially. Therefore, this report should not be relied upon after a period of three (3) years without our review nor is it applicable for any properties other than those studied.

5. Validity of the recommendations contained in this report is also dependent upon the prescribed testing and observation program during the site preparation and construction phases. Our firm assumes no responsibility for construction compliance with these design concepts and recommendations unless we have been retained to perform continuous on-site testing and review during all phases of site preparation, grading, and foundation/slab construction.

Thank you for the opportunity to have been of service in preparing this report. If you have any questions or require additional assistance, please feel free to contact the undersigned at (805) 458-9710.

Sincerely,

GSI SOILS INC.

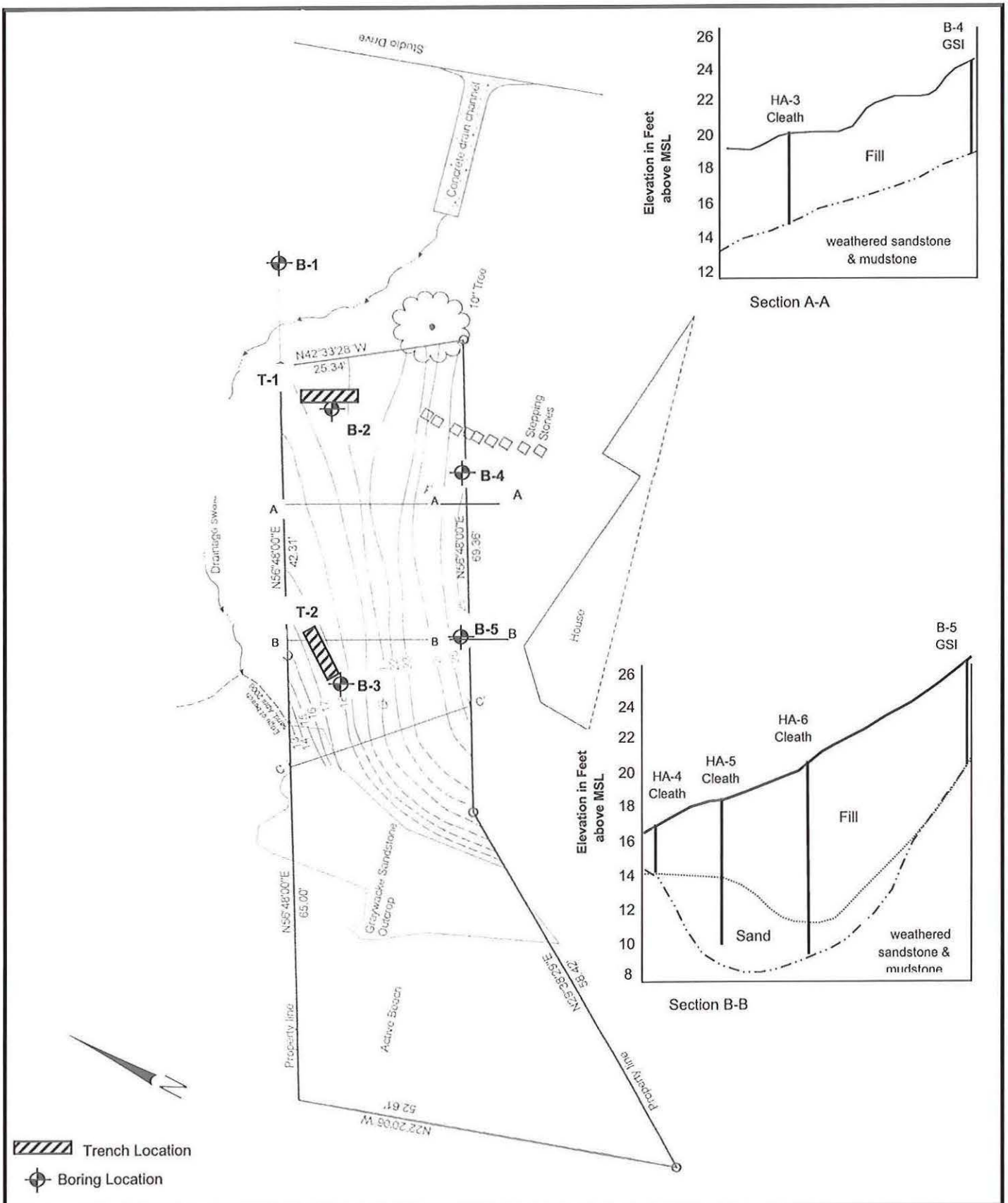
R Church

Ronald J. Church
Senior Engineer
GE #2184

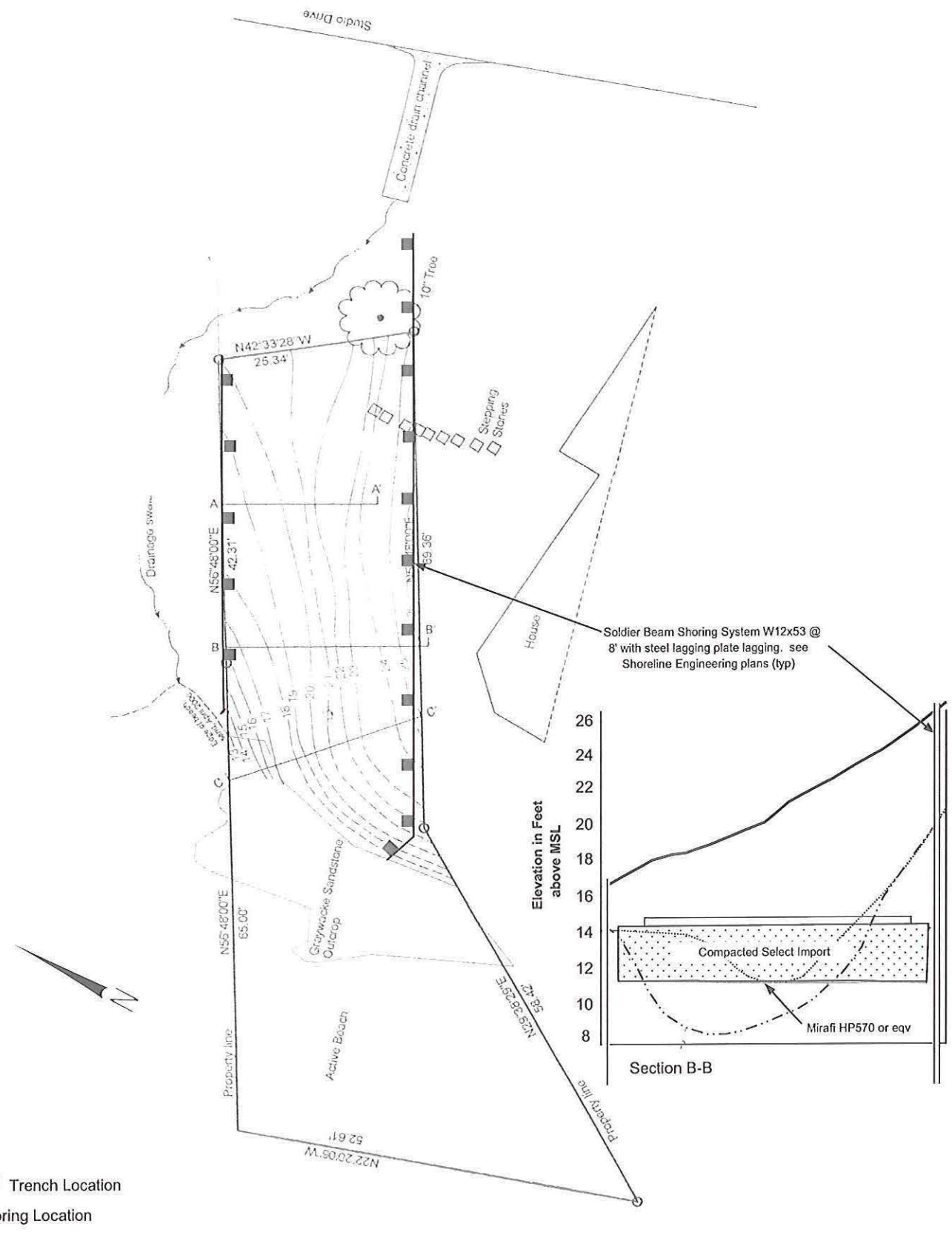
FIGURES

N

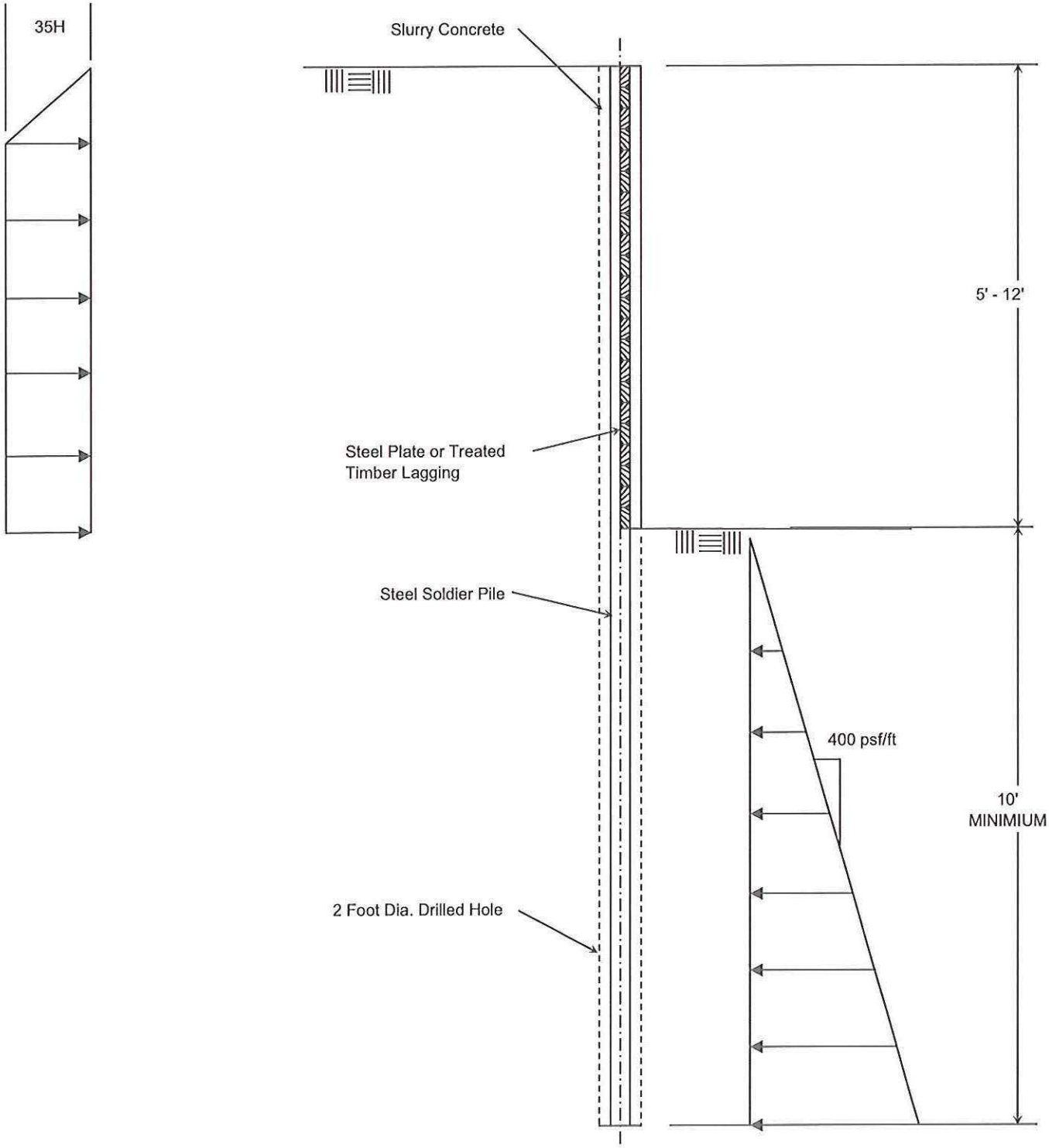
		Project No.	Figure No.
GSI	SITE MAP PROPOSED RESIDENCE LOT 41, STUDIO DRIVE CAYUCOS, CALIFORNIA	6-4210	1



		Project No.	Figure No.
GSI	SITE PLAN PROPOSED RESIDENCE LOT 41, STUDIO DRIVE CAYUCOS, CALIFORNIA	6-4210	2



		Project No.	Figure No.
GSI	SOLDIER PILE SHORING PROPOSED RESIDENCE LOT 41, STUDIO DRIVE CAYUCOS, CALIFORNIA	6-4210	3



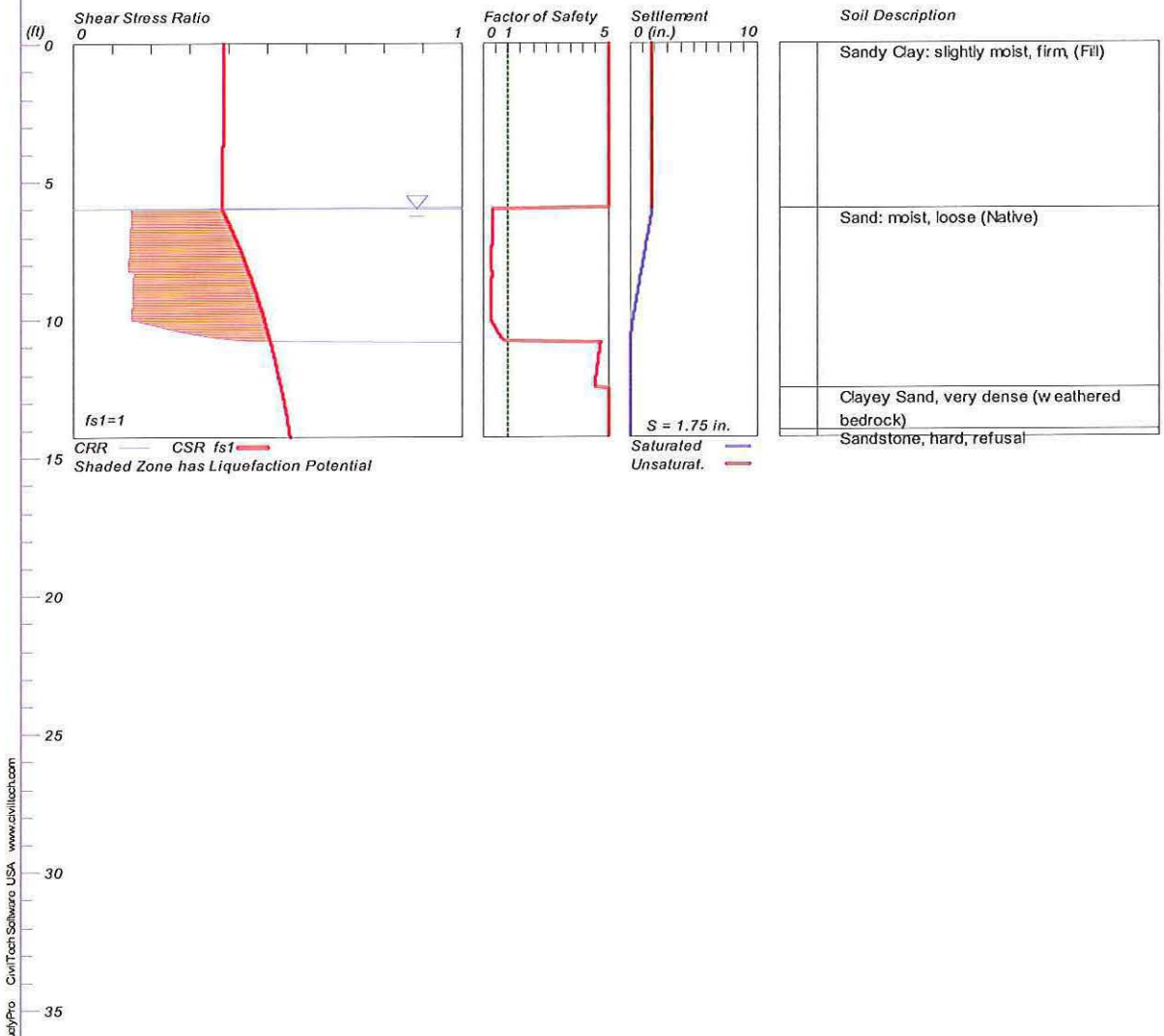
		Project No.	Figure No.
GSI	SHORING SCHEMATIC PROPOSED RESIDENCE LOT 41, STUDIO DRIVE CAYUCOS, CALIFORNIA	6-4210	4

LIQUEFACTION ANALYSIS

LOPERENA RESIDENCE

Hole No.=B-3 Water Depth=6 ft Surface Elev.=18

Magnitude=7
Acceleration=0.60g



CivilTech Corporation

LOT 41, STUDIO DRIVE, CAYUCOS

Plate A-1

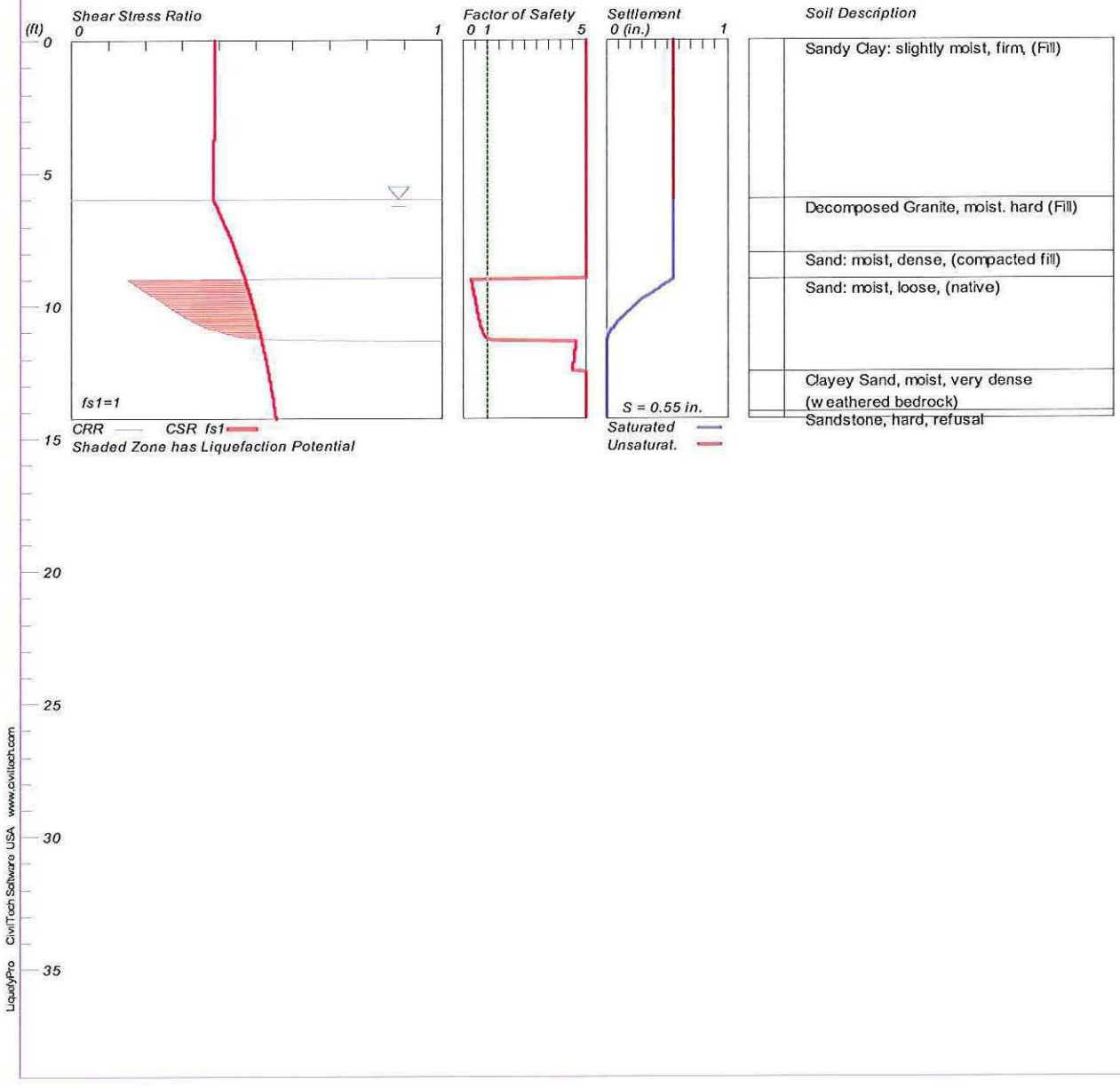
		Project No.	Figure No.
GSI	LIQUEFACTION PROPOSED RESIDENCE LOT 41, STUDIO DRIVE CAYUCOS, CALIFORNIA	6-4210	5

LIQUEFACTION ANALYSIS

LOPERENA RESIDENCE

Hole No.=B-3 Water Depth=6 ft Surface Elev.=18

Magnitude=7
Acceleration=0.60g



CivilTech Corporation

LOT 41, STUDIO DRIVE, CAYUCOS

Plate A-1

		Project No.	Figure No.
GSI	LIQUEFACTION PROPOSED RESIDENCE LOT 41, STUDIO DRIVE CAYUCOS, CALIFORNIA	6-4210	6

APPENDIX A

Field Investigation
Key to Boring and Trench Logs
Boring and Trench Logs

FIELD INVESTIGATION

Test Hole Drilling and Trench Excavations

The initial field investigation was conducted on December 12 and 19, 2006. Three (3) exploratory borings were drilled and two (2) trenches were excavated at the approximate locations indicated on the Site Plan, Figure 2. Subsequently two (2) borings were drilled on September 29 and 30, 2011. These boring locations are also indicated on Figure 2. The locations of the borings and trenches were approximated in the field.

Undisturbed and bulk samples were obtained at various depths during test hole drilling. The undisturbed samples were obtained by driving a 2.4-inch inside diameter sampler into soils. Bulk samples were also obtained during drilling.

Logs of Boring and Trenches

A continuous log of soils, as encountered in the borings and trenches was recorded at the time of the field investigation, by a Staff Engineer. The Exploration Boring and Trench Logs are attached.

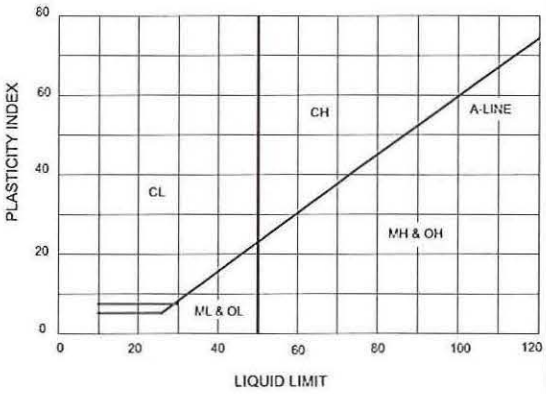
Locations and depth of sampling, in-situ soil dry densities and moisture contents are tabulated in the Boring Logs.

UNIFIED SOIL CLASSIFICATION SYSTEMS

MAJOR DIVISION			SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS Over 50% > #200 sieve	GRAVELS Over 50% > #4 sieve	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
	SANDS Over 50% < #4 sieve	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS
			SP	POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES
			SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS Over 50% < #200 sieve	SILTS AND CLAYS Liquid limit < 50		ML	INORGANIC SILTS, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY, SANDY, OR SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS Liquid limit > 50		MH	INORGANIC SILTS , MICACEOUS OR DIATOMACIOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC CLAYS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS

PLASTICITY CHART

USED FOR CLASSIFICATION OF FINE GRAINED SOILS



SOIL GRAIN SIZE

U.S. STANDARD SIEVE

	6"	3"	3/4"	4	10	40	200
BOULDERS	COBBLES	GRAVEL COARSE	GRAVEL FINE	SAND COARSE	SAND MEDIUM	SAND FINE	SILT / CLAY
150		75	19	4.75	2.0	0.425	0.075 / 0.002

SOIL GRAIN SIZE IN MILLIMETERS

SAMPLE DRIVING RECORD

BLOWS PER FOOT	DESCRIPTION
25	25 BLOWS DROVE SAMPLER 12 INCHES, AFTER INITIAL 6 INCHES OF SEATING
50/7"	50 BLOWS DROVE SAMPLER 7 INCHES, AFTER INITIAL 6 INCHES OF SEATING
Ref/3"	50 BLOWS DROVE SAMPLER 3 INCHES DURING OR AFTER INITIAL 6 INCHES OF SEATING

NOTE: TO AVOID DAMAGE TO SAMPLING TOOLS, DRIVING IS LIMITED TO 50 BLOWS PER 6 INCHES DURING OR AFTER SEATING INTERVAL

KEY TO TEST DATA

R	Bag Sample	CONS	Consolidation (ASTM D2435)
	Drive, No Sample Collected	DS	Cons. Drained Direct Shear (ASTM D3080)
	2 1/2" O.D. Mod. California Sampler, Not Tested	PP	Pocket Penetrometer
	2 1/2" O.D. Mod. California Sampler, Tested	GSD	Grain Size Distribution (ASTM D422)
	Standard Penetration Test	CP	Compaction Test (ASTM D1557)
	Sample Attempted with No Recovery	EI	Expansion Index (ASTM D4829)
	Water Level at Time of Drilling	LL	Liquid Limit (in percent)
	Water Level after Drilling	PI	Plasticity Index

RELATIVE DENSITY

SANDS, GRAVELS, AND NON PLASTIC SILTS	BLOWS/FOOT
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

RELATIVE DENSITY

CLAYS AND PLASTIC SILTS	STRENGTH	BLOWS/FOOT
VERY SOFT	0 - 1/4	0 - 2
SOFT	1/4 - 1/2	2 - 4
FIRM	1/2 - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	OVER 4	OVER 32

GSI

PROJECT NO.:	6-4210	SOIL CLASSIFICATION CHART AND BORING LOG LEGEND
DATE DRILLED:	12/6/2006	
PROPOSED RESIDENCE CAYUCOS, CALIFORNIA		FIGURE NO. A-1

LOGGED BY: **JR**	DRILL RIG: **Portable Rig**	BORING NO.: **B-1**
ELEVATION: **20'**	BORING DIAMETER (INCH): **6**	DATE DRILLED: **6 December 2006**
GROUNDWATER DEPTH (FT):		

ELEVATION (FT)	DEPTH (FT)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION	SOIL TYPE	SAMPLE	CONV. SPT BLOW COUNT	WATER CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMIT	PLASIT. INDEX	UNC. COMP. STRENGTH (PSF)	COMMENTS AND ADDITIONAL TESTS
19	1		Clayey Sand: dark brown, slightly moist, fine to coarse grained, some gravel and silt, some debris (concrete), loose (fill)	SC-CL								
18	2		decreasing clay (SC)		B		8.0					
17	3					7	7.4	98.5				
16	4											
15	5		Clayey Sand: yellow brown, slightly moist, fine to medium grained, trace silt, dense	SC								
14	6				B		7.8					
13	7		Boring terminated, refusal at 7.0 feet (50 blows for <2 inch)									
12	8											
11	9											
10	10											
9	11											
8	12											
7	13											
6	14											
5	15											
4	16											
3	17											
2	18											
1	19											
0	20											

EXPLORATORY BORING LOGS

GSI

PROPOSED RESIDENCE
LOT 41, STUDIO DRIVE

PROJECT NO.	DATE	FIGURE NO.
6-4210	**December-11**	**A-2**

LOGGED BY: **JR**	DRILL RIG: **Portable Rig**	BORING NO.: **B-2**
ELEVATION: **20'**	BORING DIAMETER (INCH): **6**	DATE DRILLED: **6 December 2006**
GROUNDWATER DEPTH (FT):		

ELEVATION (FT)	DEPTH (FT)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION	SOIL TYPE	SAMPLE	CONV. SPT BLOW COUNT	WATER CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMIT	PLASIT. INDEX	UNC. COMP. STRENGTH (PSF)	COMMENTS AND ADDITIONAL TESTS
19	1		Clayey Sand: dark yellow brown, slightly moist, fine to coarse grained, some gravel and silt, some debris (asphalt, concrete),	SC-CL	B							
18	2		Silty Sand: yellow brown, slightly moist, fine to medium grained, some gravel, some debris (asphalt, plastic), loose (fill)	SM-SC	B		4.3					
17	3					7	7.1					
16	4		trace to some clay		B							
15	5		Sandy Clay: grayish brown/olive gray, slightly moist, fine to medium grained, trace silt, stiff to very stiff	CL								
14	6					28	8.9					EI = 92
13	7											
12	8				B		5.3					
11	9		Boring terminated, refusal at 9.0 feet (50 blows for <1 inch)									
10	10											
9	11											
8	12											
7	13											
6	14											
5	15											
4	16											
3	17											
2	18											
1	19											
0	20											

EXPLORATORY BORING LOGS

GSI

PROPOSED RESIDENCE
LOT 41, STUDIO DRIVE

PROJECT NO.	DATE	FIGURE NO.
6-4210	**December-11**	**A-3**

LOGGED BY: **JR**	DRILL RIG: **Portable Rig**	BORING NO.: **B-3**
ELEVATION: **18'**	BORING DIAMETER (INCH): **6**	DATE DRILLED: **6 December 2006**

GROUNDWATER DEPTH (FT):

ELEVATION (FT)	DEPTH (FT)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION	SOIL TYPE	SAMPLE	CONV. SPT BLOW COUNT	WATER CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMIT	PLASIT. INDEX	UNC. COMP. STRENGTH (PSF)	COMMENTS AND ADDITIONAL TESTS
17	1		Sandy Clay: reddish brown, slightly moist, fine to coarse grained sand, some gravel and silt, some debris (asphalt), firm (fill)	CL								
16	2				B		11.9					
15	3		Sandy Clay with Gravel and Cobbles: dark grayish brown, slightly moist, some debris (asphalt), firm (fill)	CL		9	5.7	101.4				
14	4		Sandy Clay: yellowish brown, slightly moist, some gravel, loose (fill)	CL	B		8.5					
13	5											
12	6		Sand: brown, moist, fine to medium grained, trace gravel, shell fragments, loose (native)	SP								
11	7				B		6.5					
10	8											
9	9					7	5.3					
8	10		some gravels									
7	11		trace gravels, very moist									
6	12				B							
5	13		Clayey Sand: yellow brown, moist, very dense	SC								
4	14				B		10.3					
			Sandstone: yellow brown, moist, hard									
3	15		Boring terminated, refusal at 14.25 feet									
2	16											
1	17											
0	18											
-1	19											
-2	20											

EXPLORATORY BORING LOGS

GSI

PROPOSED RESIDENCE
LOT 41, STUDIO DRIVE

PROJECT NO.	DATE	FIGURE NO.
6-4210	**December-11**	**A-4**

LOGGED BY: **MS**	DRILL RIG: **Power Auger**	BORING NO.: **B-4**
ELEVATION: **25'**	BORING DIAMETER (INCH): **4**	DATE DRILLED: **29 September 2011**

GROUNDWATER DEPTH (FT):

ELEVATION (FT)	DEPTH (FT)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION	SOIL TYPE	SAMPLE	CONV. SPT BLOW COUNT	WATER CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMIT	PLASIT. INDEX	UNC. COMP. STRENGTH (PSF)	COMMENTS AND ADDITIONAL TESTS
24	1		Sandy Clay: reddish brown, moist, fine to medium grained sand, trace gravel and silt, firm (fill)	CL	B		16.8					
23	2		Sandy Silty Clay: mottled reddish brown & dark brown, moist, some gravel, firm (fill)	CL	B		21.3					
22	3		Sandy Clay: dark greyish brown, moist, some gravel, stiff (fill)	CL		15	12.3	103.4				
21	4				B		10.4					
20	5		Sandy Clay: greyish brown, moist, some rock fragments, very stiff	CL		47	11.9	112.3				
19	6		Clayey Sandstone: olive brown, moist, very dense	SC	B							
18	7											
17	8		refusal			50/1"	8.7					
			Boring terminated, refusal at 8 feet									
16	9											
15	10											
14	11											
13	12											
12	13											
11	14											
10	15											
9	16											
8	17											
7	18											
6	19											
5	20											

EXPLORATORY BORING LOGS

GSI

PROPOSED RESIDENCE
LOT 41, STUDIO DRIVE

PROJECT NO.	DATE	FIGURE NO.
6-4210	**December-11**	**A-5**

LOGGED BY: **MS**	DRILL RIG: **Power Auger**	BORING NO.: **B-5**
ELEVATION: **25'**	BORING DIAMETER (INCH): **4**	DATE DRILLED: **29 September 2011**

GROUNDWATER DEPTH (FT):

ELEVATION (FT)	DEPTH (FT)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION	SOIL TYPE	SAMPLE	CONV. SPT BLOW COUNT	WATER CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMIT	PLASIT. INDEX	UNC. COMP. STRENGTH (PSF)	COMMENTS AND ADDITIONAL TESTS
24	1		Sandy Silty Clay: mottled reddish brown & dark brown, moist, trace gravel, firm (fill)	CL	B							
23	2				B		20.2					
22	3		Sandy Clay: dark brown, moist, some gravel & sand, stiff (fill)	CL		11	19.7	100.7				
21	4				B							
20	5		Sandy Clay: greyish brown, moist, some rock fragments, very stiff	CL								
19	6					39	9.3	110.2				
18	7				B							
17	8		Clayey Sandstone: olive brown, moist, very dense	SC								
16	9		refusal			50/2"	10.4					Comp Str = 1240 psi
15	10		Boring terminated 9.5 feet									
14	11											
13	12											
12	13											
11	14											
10	15											
9	16											
8	17											
7	18											
6	19											
5	20											

EXPLORATORY BORING LOGS

GSI

PROPOSED RESIDENCE
LOT 41, STUDIO DRIVE

PROJECT NO.	DATE	FIGURE NO.
6-4210	**December-11**	**A-6**

LOGGED BY: **GM**	DRILL RIG: **Backhoe**	BORING NO.: **T-1**
ELEVATION: **20'**	BORING DIAMETER (INCH):	DATE DRILLED: **19 December 2006**
GROUNDWATER DEPTH (FT):		

ELEVATION (FT)	DEPTH (FT)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION	SOIL TYPE	SAMPLE	CONV. SPT BLOW COUNT	WATER CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMIT	PLASIT. INDEX	UNC. COMP. STRENGTH (PSF)	COMMENTS AND ADDITIONAL TESTS
19	1		Clayey Sand: dark brown, slightly moist, fine to coarse grained, some gravel and silt, debris (conrete, asphalt), loose (fill)	SC-CL								
18	2		Silty Sand: light yellow brown, slightly moist, fine to coarse grained, some gravel, some debris (asphalt, plastic), loose (fill)	SM-SC	B		5.2					
17	3											
16	4		plastic at 4.5'									
15	5		Sandy Clay: grayish brown, slightly moist, fine to medium grained, trace gravel and silt, firm	CL								
14	6				B		9.6					
13	7				B		7.5					
12	8		Excavation terminated at 7.5 feet in hard material									
11	9											
10	10											
9	11											
8	12											
7	13											
6	14											
5	15											
4	16											
3	17											
2	18											
1	19											
0	20											

EXPLORATORY BORING LOGS

GSI

PROPOSED RESIDENCE
LOT 41, STUDIO DRIVE

PROJECT NO.	DATE	FIGURE NO.
6-4210	**December-11**	**A-7**

LOGGED BY: **GM**	DRILL RIG: **Backhoe**	BORING NO.: **T-2**
ELEVATION: **18'**	BORING DIAMETER (INCH):	DATE DRILLED: **19 December 2006**
GROUNDWATER DEPTH (FT):		

ELEVATION (FT)	DEPTH (FT)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION	SOIL TYPE	SAMPLE	CONV. SPT BLOW COUNT	WATER CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMIT	PLASIT. INDEX	UNC. COMP. STRENGTH (PSF)	COMMENTS AND ADDITIONAL TESTS
17	1		Sandy Clay: reddish brown, slightly moist, fine to coarse grained sand, some gravel and silt, debris (asphalt), firm (fill)	CL	B		10.6					
16	2		with Gravel and Cobbles: dark grayish brown, (fill)		B		5.1					
15	3											
14	4		Sandy Clay: yellowish brown, slightly moist, some gravel, some debris (bottles), loose (fill)	CL								
13	5											
12	6		Sand: brown, moist, fine to medium grained, trace gravel, shell fragments, loose (native)	SP	B		9.8					
11	7											
10	8											
9	9				B		2.6					
8	10		Excavation terminated at 10 feet									
7	11											
6	12											
5	13											
4	14											
3	15											
2	16											
1	17											
0	18											
-1	19											
-2	20											

EXPLORATORY BORING LOGS

GSI

PROPOSED RESIDENCE
LOT 41, STUDIO DRIVE

PROJECT NO.	DATE	FIGURE NO.
6-4210	**December-11**	**A-8**

APPENDIX B

Moisture-Density Tests
Direct Shear Test
Expansion Index Test
Consolidation-Pressure Tests

LABORATORY TESTING

Moisture-Density Tests

The field moisture content, as a percentage of the dry weight of the soil, was determined by weighing samples before and after oven drying. Dry densities, in pounds per cubic foot, were also determined for the undisturbed samples. Results of these determinations are shown in the Exploration Boring Logs.

Direct Shear Test

Direct shear tests were performed on undisturbed samples, to determine strength characteristics of the soil. The test specimens were soaked prior to testing. Results of the shear strength tests are attached.

Expansion Index Tests

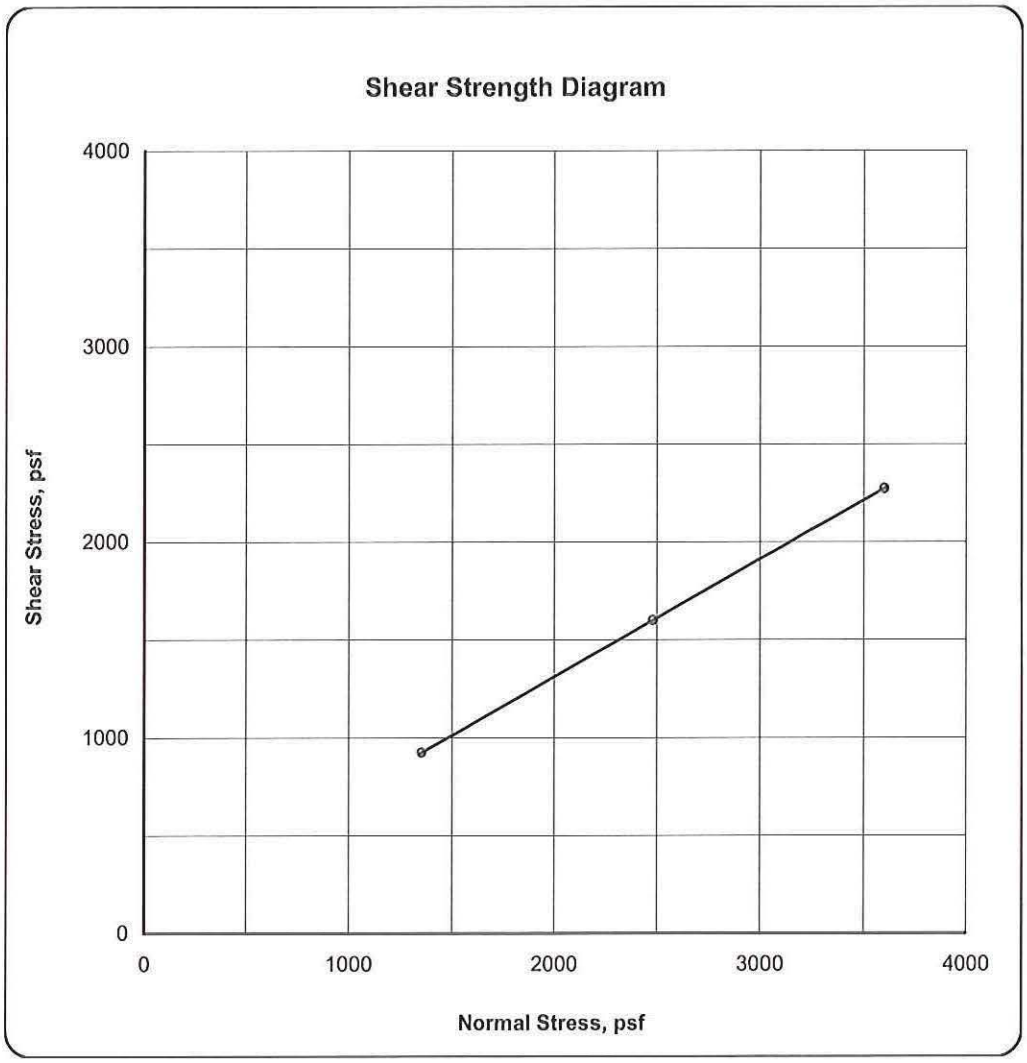
An expansion index of 92 was obtained for the sandy clay soils encountered in boring B-2. The test procedures were performed in accordance with Uniform Building Code Standard 29-2.

Consolidation-Pressure Tests

Consolidation characteristics of potentially compressive native soils were determined by using undisturbed soil specimens subjected to dead weight loading increments in a consolidometer. The samples were wetted when loading reached their approximate overburden pressure. Test results are illustrated by a curve, indicating the percent volume change of the soil, under various loads. Results of the Consolidation-Pressure test are attached.

DIRECT SHEAR TEST

ASTM D3080-90 (Modified for unconsolidated-undrained conditions)



Project:	PROPOSED RESIDENCE	Project No.	6-4210
Sample Location:	B-1 @ 3 feet	Initial Dry Density (pcf)	98.5
Soil Description:	**Clayey Sand**	Initial Moisture (%)	7.4
Sample Type:	○ Remolded ◉ Ring	**Peak Shear Angle** **Cohesion (psf)**	**31** **110**

CONSOLIDATION TEST
ASTM D2435

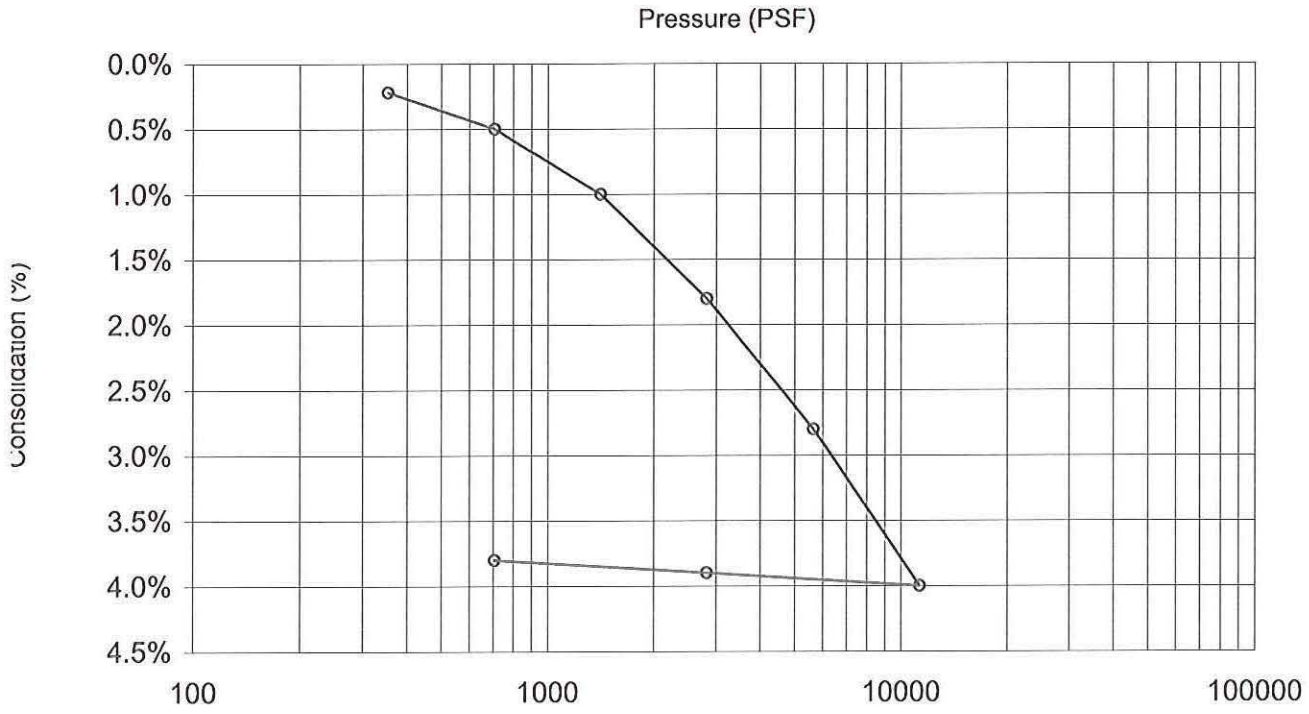
DATE: 12/22/06

SOIL TYPE: Sandy Clay

LOCATION: B-2 @ 6 Feet

PROJECT: Lot 41, Studio Drive

SITE LOCATION: Cayucos, California



GSI SOILS INC.

524 East Chapel Street
Santa Maria, CA 93454
Tel: (805) 349-0140
Fax: (805) 349-8861

October 1, 2012
Proposal 6-4210

Bruce Elster
Shoreline Engineering
505 Harbor Street
Morro Bay, CA 93442

Subject: Response to Supplemental Geotechnical Peer Review
Loperena Residence
Lot 41, Studio Drive
Cayucos, California

Reference: Updated Geotechnical Investigation, Proposed Residence, Lot 41, Studio Drive, Cayucos, California, by GSI Soils Inc, dated December 27, 2011, Project No. 6-4210.

Dear Bruce:

As requested, the following responses are provided to the supplemental Geotechnical Peer Review Letter (dated August 21, 2012) prepared by Cotton, Shires & Associates Inc (CSA).

Comment #1 – Bedrock Compressive Strength: Due to the hard bedrock conditions the sample obtained from Boring B-5 was of insufficient size for testing in compliance with ASTM Test Methods. As indicated on the attached test sheet the sample was cut to a 1 inch square to create a 2:1 height to width ratio. The sample was tested in a multi-loader with no confining stress and tested to failure. The test was performed to determine an approximate strength of the underlying bedrock material. As indicated on the test sheet a modified version of ASTM D7012 – C would be the test most representative of the method used. However, the testing was primarily performed to show that the underlying bedrock is hard and would have negligible potential for settlements from the applied loading.

Comment #2 – Site Grading: The site will be graded in accordance with Section 5.2 of the referenced report and no scarification will be required. As indicated in Section 5.2 where native sands are exposed a layer of geofabric (Mirafi HP570 or equivalent) should be placed across the surface. A minimum of 3 feet of select fill materials (decomposed granite or Class II Base) should then be placed and compacted to 90 percent of ASTM D1557-02 to pad grade.

Comment #3 – Liquefaction: The term low category (for liquefaction) refers to the potential settlements (1.75") and effects on the proposed structure. In our opinion, a liquefaction potential

in the low category would generally result in minor to cosmetic damage to the structure. With the proposed site grading total settlements would be around ½ inch. This amount of settlement is considered tolerable for the proposed structure and would be indicative of liquefaction in the negligible category.

Comment #4 – Skin Friction Values: As indicated in the boring logs, drilling at the site was performed using a power auger. Penetration into the bedrock was very slow and required substantial down pressure. Subsequent driven samples were obtained with difficulty due to the hardness of the bedrock. As indicated in the boring logs, refusal was obtained in 1 to 2 inches of penetration into the harder bedrock. GSI would estimate that the shear angle for the bedrock would exceed 45 degrees with substantial cohesion. The skin friction values (2000 psf downward and 1600 psf upward) would be considered relatively conservative for the materials encountered.

Comment #5 – Soldier Piles: It is our understanding that the soldier piles will be removed. Considering the soils types (sandy clays over bedrock) we would anticipate minimal to no settlements in the adjacent materials and to the residence located to the south.

Comment #6 – Pavement Areas: In general the top lift of trench backfill under pavement areas should be scarified to a depth of 12 inches. The soil should then be moisture conditioned to slightly above optimum moisture content and compacted to a minimum of 90 percent of maximum dry density. The upper 6 inches of subgrade beneath all paved areas should then be compacted to at least 95 percent relative compaction. Subgrade soils should not be allowed to dry out or have excessive construction traffic between the time of water conditioning and compaction, and the time of placement of the pavement structural section.

Please feel free to call me at (805) 458-9710 if you have any questions.

Sincerely,
GSI SOILS INC.

R Church

Ron Church
GE 2184

Cc Michael Phipps/David Schrier, Cotton, Shires & Associates

UNCONFINED COMPRESSIVE STRENGTH

Test Method: ASTM D7012 - C (modified by GSI)

Job #:	6-4210	Job Name:	Loperena Residence
Lab Job #:	787	Client:	Loperena

SAMPLE IDENTIFICATION								
Boring Number:	B-5							
Sample Number:	B5-9							
Depth:	9'							
Moisture Condition:	moist							
Break Date:	10/13/11							
Sample Description:	Clayey Sandstone, olive brown, very dense							

SAMPLE DIMENSIONS and UNIT WEIGHT								
Mass of Sample (g):								
Length (in):	2.00							
Diameter 1 (in):	1.00							
Diameter 2 (in):	1.00							
Avg. Diameter (in):	1.000							
Volume, ft^3	0.000909026							
L/D Ratio:	2.00							
Total Unit Weight (lbs/ft^2):	**0.00**							

STRENGTH DATA								
Maximum Load (lbs):	965							
Compressive Strength (psi):	1229							
Strength Corrected for	**1239.83**							

Tested By:	MM	Date:	10/13/2011	Checked By:	RC