January 24, 2011

TO: Deanna Christensen
Jack Ainsworth

FROM: Lesley Ewing

SUBJECT: Engineering Review of the Sweetwater Mesa Project

I have been asked to review the engineering aspects of the proposed Sweetwater Mesa Project, including the access road, driveways and building pads. Attachment 1 includes the full list of documents that I have reviewed. In addition to reviewing the submitted materials, I visited the site on 28 January 2010 and have participated in numerous conference calls and meetings with staff, technical consultants, and the applicants' consultants concerning this proposed project.

The proposed project will be located in the Santa Monica Mountains and will include an access road, utilities, and building pads, drive ways, septic systems, and ancillary buildings for 5 separate home sites. The access road is an extension of Sweetwater Mesa Road; part of the road would be in the City of Malibu and part of the road would be in unincorporated Los Angeles County. My review only covers the portion of the road within unincorporated Los Angeles County. This area of the Santa Monica Mountains is quite rugged, and the current roadway is a dirt trail only easily accessible by four-wheel drive. To underscore the steepness of the terrain, during our site visit, the four-wheel drive vehicles could only drive safely on the lower part of the road, and we were only able to get to the steeper, northern (upper) part of the site on foot.

There are several large landslides on the site, and the geologic conditions pose significant engineering challenges to provide safe development, especially for the access road. In addition to the basic access requirements for a road (providing ingress and egress for construction equipment, building residents and guests, fire equipment, etc.), the County will require that, at a minimum, roadway be designed to remain stable in the event of landslide movement. And, it must stabilize the landslide material upslope of the road. During my review of this project, three different structural engineering designs have been developed and proposed for the roadway.

The portion of the access road within the unincorporated County will be 4,883 feet long or approximately 0.9 miles long. It will cross two large landslides, and two sections of the road, one 590 feet long and one 905 feet long, will be supported on caissons to provide for safe access across these slide areas. In addition to the 1,495 feet of caisson-supported roadway, there will be several retaining walls and a significant amount of cut and fill to provide for a level road surface. The civil engineering plans for stabilizing the road would include, in total 5 retaining walls ranging in length from 90 feet to 390 feet and totaling 955 feet of retaining wall. The retaining walls would range in height from averages of 5 to 11 feet and maximum heights of 7.5 to 18 feet. The longest retaining wall, along the right side (or upslope side) of the northern portion of the road, has been designed to be 390 feet long and to have an average height of 11 feet and a

Exhibit 26
CDP 4-10-040 through 4-10-045
Lesley Ewing Memorandum

maximum height of 18 feet. Due to the dimensions of the retaining walls, it is quite possible that they could be visible from public vantage points. If the project is approved, it is suggested that these walls should be colored, texturized and possibly vegetated so that they will blend in visually with the surrounding area.

The road will also require approximately 20,100 cubic yards of cut, 32,950 cubic yard of fill, and 294,150 square feet of disturbed land area (6.75 acres). There will be several sections of finished road that will be quite steep. There will be sections approximately 998 feet long, 1,085 feet long and 535 feet long that will have a grade of 18.95% and there will be one additional 285 foot long section that will have a grade of 17.25%. These steep grade sections do not connect; each section will be separated by stretches of road that are at a much gentler grade.

The initial engineering design proposed to place the road on a combination of deep caissons and "dog bone" or double-barreled caissons. The reinforcing steel for each caisson and each "dog bone" caisson was designed to be oriented to the main direction of the slide at each caisson site. The project design developed about a dozen main caisson template designs, and each installed caisson would use one of these dozen main templates, with careful individual fabrication and installation for the exact slide conditions at each caisson location. The main variations for the caisson designs related to diameter, length, extent of reinforcing throughout the caisson and orientation of the reinforcing frame. The caisson road support was a rather complex structural engineering system. It was a type of system that I had never seen before, and review of this system required structural engineering that was outside my area of expertise. To fill in this needed expertise, the consulting firm, Cotton, Shires and Associates (CSA), was hired to assist our review of this project. Attachments 2, 3, 4, and 5 provide the initial scope of work for CSA, their initial project review, the amended scope of work and the additional project review. In addition to providing technical review of the project, I served as the main contact between CSA, Commission staff and the applicants' technical consultants.

Engineering is one of the last steps in the project development process. The earlier steps are to characterize the site geology and soil strength parameters. The initial roadway engineering design was developed from soils information developed through the initial site characterization. As a result of the site visit and project review by CSA, an additional large diameter bore hole (B-38) was logged and several test pits at the upper part of the proposed road (near the border with the City of Malibu) and trenches (near the proposed home site for CDP 4-10-043) were excavated and logged. Additional information on the site conditions and the slide plane was developed through these field efforts. Samples of the slide plane material were taken from boring B-38; this sample was eventually tested and the test results added to the available information about the conditions of the site.

In addition to issues concerning the geologic and geotechnical characterization of the site, the review by CSA noted a number of corrections or clarifications that were needed for the structural engineering calculations and noted that "the Consultant has designed the piles without applying the code FOS [Factor of Safety], instead using a 'load factor' equal to 1.067 or the ratio of 1.6 (code structural FOS) over 1.5 (SSA [Slope Stability Analysis] FOS). This is not typically accepted design practice since these two factors of safety may not necessarily be redundant (one applies to uncertainties in strengths, distributions and behaviors of structural materials and the other applies to uncertainties in subsurface conditions, soil parameters and limitation inherent in slope stability analyses, etc.).” (CSA 2010, page 16)

The March 8, 2010 CSA letter concluded that, “By refining the geologic landslide mapping, it is our preliminary opinion that some reductions in the amount and size of the stabilization elements could be realized. It also appears that with some modifications to the roadway alignment, some of the landslide crossings could be either eliminated or reduced, which would reduce the extent of subsurface stabilization elements needed.” (CSA 2010, page 25) This Review Letter further recommended that the “geologic characterization of the site needed to be refined”, and the geotechnical engineering consultant needed to “perform supplemental laboratory testing to better determine landslide-specific shear strengths” and the “civil and structural engineering consultants will then need to address the refined geologic characterization and geotechnical engineering analysis of that refined characterization utilizing approved design practices.” (CSA 2010, page 26) Following extensive discussions on the site conditions and examination of soils tests and potential slide geometries, soil strength parameters of 15-degree friction angle and 200-psf cohesion were calculated from back calculations and all parties agreed to use these parameters for design purposes. The currently-proposed road stabilization design uses these strength parameters.

The proposed road support system has been through three different design iterations. The initial design was the combination of cylindrical caissons and “dog bone” caissons. In early June 2010 we were provided with a second road support design, which relied upon traditional cylindrical caissons for the entire road support system, and the “dog bone” caissons had been deleted. As with the initial design, the caissons would require careful field installation since reinforcing steel for each caisson was designed to be oriented with the direction of the slide.

The third and currently proposed design was prepared in November 2010 (dated 11/16/2010). This road design is in the same road easement as the previous two designs and, like the June 2010 design option, it uses cylindrical piles for the roadway that crosses the landslide areas. In this design option, the road will be supported on 123 large diameter reinforced concrete caissons. An additional fourteen (14) 5-foot diameter caissons for rock fall protection will be installed at the southern portion of the road, close to the City of Malibu boundary. The main road support system will use caissons ranging in diameter from 2 to 5 feet, with lengths up to 79 feet. The reinforcing steel in each caisson has been designed to act along the main axis of the slide, thus, like the two earlier designs, the steel must be installed to orient in the direction of the slide plane. At present 8 caissons are shown as being less than 20 feet long; however, the applicants' structural engineering consultant has noted that for the final plans all caissons will be at least 20 feet long so these 8 caissons should be changed in the final plans to provide for this additional embedment length. Of the 20,100 cubic feet of cut that is needed for the roadway, almost 25% or 4,850 cubic yards will be cut material excavated for installation of the caissons.

For alternatives analysis, the applicants' structural engineer examined the option of a tied-back wall rather than a caisson support (LC Engineering, September 2, 2010). Such a design was considered since it was thought to have the potential to further reduce both the caisson diameter and necessary reinforcing steel. However, the assessment of this option found that the tie-back installation would require far more site disturbance than the caissons, since large trenches would need to be excavated downslope of the slide to install the tiebacks. Approximately 1,010 feet of roadway would require slot excavations at least 30 to 60 feet deep to install the tie-back system, extending the site disturbance well beyond the existing roadway footprint. I have reviewed this analysis and concur that a tie-back stabilization system at this site would cause greater site disturbance than the caissons.

The roadway alignment has been closely constrained by the allowed road easement. No alternative alignments were examined due to the limitations posed by the allowable easement. In the second review letter by Cotton Shires, dated January 21, 2011, they stated that, "By refining the geologic landslide mapping, reductions in the amount and size of stabilization elements have been realized. It appears because of the steepness of the roadway corridor, the ability to devise alternative designs is limited." (CSA 2011, page 16)

The analysis of the currently-proposed caisson design is included in the January 21, 2011 report from CSA. This analysis concludes that the proposed road stabilization is "a reasonable approach to address these challenges [i.e. the site terrain and geology] given the site characterization and analyses performed. Consequently, it is our opinion that, with the exception of some of our more minor concerns and structural details, the applicant's consultants have satisfactorily addressed the comments of our previous report dated March 8, 2010 and have satisfactorily performed their work within the standard of care of their respective disciplines." (CSA 2011, page 16)

I have carefully reviewed the applicants' material as well as the CSA review, and I concur in CSA's engineering review of the most recent 5-foot diameter caisson design option. There remain some aspects of the caisson design that are itemized in the CSA letter, on pages 10 and 11. These design aspects need to be addressed by the applicants' structural engineer before the design can be approved. None of these concerns raises issues that would result in a fatal flaw to the design, but the design cannot be considered finished until these concerns are addressed. For example, guardrails are an important safety element for steep mountain roads; design details need to be included in the main design since the impact loads experienced by the guardrails can be high and should not be considered an after thought to the main design. The applicants' civil and structural engineers have been aware of the CSA design concerns for several months. However, since the County of Los Angeles may raise additional design details during its project review, the applicants' civil and structural engineers have indicated that they would prefer to address our design issues at the time they address those raised by the County. Since the changes required by the CSA review would not alter the coastal resource impacts from the proposed project, there is no reason to delay project review for these changes. However, if the project is approved, final approval cannot occur until these changes are made.

In addition to the design changes itemized on pages 10 and 11 of the January 21, 2011 report, CSA has provided 3 significant recommendations that should be met – (1) that the reinforcing steel be designed for 20 to 30 degrees of uncertainty in the main direction of the slide force (10 to 15 degrees on each side) (2) that the caisson designs be checked for compliance with the California Building Code (equation 9-7) and the American Concrete Institute (ACI) guidelines (Section 9.2.1), and (3) that a geologist be on-site to inspect the installation and verify proper orientation of each caisson. These recommendations will help insure that the proposed road support and slide mitigation will perform as intended. If the proposed project is recommended for approval, it should be conditioned to include these three recommendations -- for the reinforcing steel to include a 30 degree uncertainty in the direction of the slide force, to check the caisson design for compliance with the California Building Code and recommendations of ACI, and to have a geologist on-site to inspect each caisson excavation and the orientation of each caisson during installation.

CSA examined and spot-checked the cut and fill volumes, lengths of retaining wall support and caisson supported roadways and determined that the applicants' consultants have correctly characterized these project aspects and necessary site disturbance. CSA has recommended that all the fill slopes be keyed and benched, even if they are not intended for future development, and this should be required for the final plan approval. With the additional design and construction recommendations from CSA and staff, the road mitigation, once completely designed, should be able to provide a safe and stable access road.

The main focus of the geologic, geotechnical and engineering review has been on the roadway since the access road crosses two large landslides. The purpose of the road is to provide access to five building sites (driveways, homes, pool areas, septic systems, etc.). The proposed home sites are placed at or near the ridgelines. Four of the home sites are outside the identified slide areas; however, the Lunch development area (CDP 4-10-040) is overlain by landslide debris that will be removed as part of the site development or mitigated by the foundation design for the house. The area proposed for development on CDP 4-10-043 (Morleigh) has a hummocky terrain and during the 28 January 2010 site visit and subsequent discussion, additional site testing was recommended for this area. Trenching across this site made clear that there was no underlying landslide mass at this site. The project has shown that the five proposed building envelopes will be or can be made structurally stable for the proposed development. There may be other locations on the property that would be able to support the proposed development; but, no analysis of alternative building sites was undertaken. A requirement of the applicants for their willingness to fund the CSA review was that the CSA analysis of alternatives should not examine alternative building sites (Attachment 2)

The five building sites will require an additional 26,250 cubic yards of cut, 1,800 cubic yards of fill and result in 2.5 acres of surface disturbance. Each home site will require between 0.22 and 0.23 acres for the residential unit and (except for the northernmost home) from 0.25 and 0.45 acres for site access from the main road. The northernmost home will have access directly off the main road and will not have additional impacts for access. There will be an additional 1.88 acres of site disturbance for contour grading of up to 13,950 cubic yards of excess fill from the project. (As noted earlier, this contour grading should be designed with a proper key, benching and drainage controls, even though it is not intended for use as a development area.) Table 1 provides a summary of the cut, fill and land disturbance attributable to each homesite and to the additional project needs, such as the LA County Fire Department turn-outs, the exceed fill and the road caissons.

In summary, the geologic hazards cannot be avoided or eliminated with the proposed development sites and identified road easement, and no analysis was undertaken to determine if hazards could be reduced through alternative easement alignments or development sites. However, treating the proposed road alignment and residential development areas as fixed, there are engineering options that will allow the proposed development to be undertaken in a manner that will minimize the risks from the identified geologic hazards. If approved, there should be conditions on this project for the following:

- Any necessary retaining walls should be colored, texturized and possibly vegetated so that they will be visually compatible with the surrounding area
- Final engineering plans should incorporate all recommendations from the CSA letter of January 21, 2011, and outlined on pages 10 and 11
- All road stabilization caissons should be at least 20 feet long, or at the length identified by the structural engineering plans
- All fill slopes and contour grading areas, including the non-structural fill areas, should be properly keyed and benched and designed to control both sub-grade and surface drainage in a non-erosive manner.
- The reinforcing steel for the caissons in the road support system should include a 30 degree uncertainty in the direction of the slide force
- The caissons for the road support system should be checked to insure compliance with the California Building Code for structural loading (Equation 9-7) and guidance by the American Concrete Institute (Section 9.2.1)
- There shall be a geologist on-site during construction of the road support system to inspect each caisson excavation and the orientation of each caisson during installation.

With the above listed modifications, the proposed project should be able to assure stability and structural integrity to a reasonable degree and to minimize risks to life and property, consistent with Coastal Act section 30253.

Please contact me if there are other aspects of this project that you would like to discuss.

Table 1: Cut, fill and Site Disturbance Associated with Each Home Site

Property	Cut (CY)	Fill (CY)	Disturbance (sq ft)	Disturbance (acres)
VERA				
Access Road				
Private Access	5,300		14,000	0.32
Development area	5,400		9,695	0.22
VERA Sub-total	*10,700*		*23,695*	*0.54*
LUNCH				
Access Road	4,800	5,950	104,900	2.41
Private Access	650	800	19,500	0.45
Development area	3,350		9,950	0.23
LUNCH Sub-total	*8,200*	*6,750*	*134,350*	*3.09*
MORLEIGH				
Access Road	9,350	3,100	69,950	1.61
Private Access	3,700	600	15,250	0.35
Development area	1,300		9,950	0.23
MORLEIGH Sub-total	*14,350*	*3,700*	*95,150*	*2.19*
MULRYAN				
Access Road	900	1,750	24,600	0.56
Private Access	1,300		10,750	0.25
Development area	1,600	400	9,550	0.22
MULRYAN Sub-total	*3,800*	*2,150*	*44,900*	*1.03*
RONAN				
Access Road	200	12,500	43,000	0.99
Private Access				
Development area	3,650		9,880	0.23
RONAN Sub-total	*3,850*	*12,500*	*52,880*	*1.22*
PILES/CAISSONS	4,850			
LACFD Staging areas		10,000	51,700	1.19
Grading of excess fill		13,950	81,750	1.88
TOTALS (1 and 2)	**46,350**	**48,700**	**484,425**	**11.12**

(1) Cut and fill totals are not equal due to volume estimates and assumptions made for bulking.
(2) Square feet of disturbance estimates sum; however, acreage values do not sum due to rounding.

Attachment 1: Reviewed Documents and Drawings

Documents and Drawings:

CalWest Geotechnical Engineering Consultants, May 25, 2007, Geotechnical Engineering Report, Proposed Custom Single-Family Residential Development, APN 4453-005-037 (Lunch), Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, May 25, 2007, Geotechnical Engineering Report, Proposed Custom Single-Family Residential Development, APN 4453-005-018 (Vera), Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, June 1, 2007, Geotechnical Engineering Report, Proposed Custom Single-Family Residential Development, APN 4453-005-092 (Mulryan), Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, June 4, 2007, Geotechnical Engineering Report, Proposed Custom Single-Family Residential Development, APN 4453-005-091 (Morleigh), Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, October 2, 2007, Geotechnical Engineering Report, Proposed Custom Single-Family Residential Development, APN 4453-005-038 (Ronan), Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, December 20, 2007, Geotechnical Engineering Addendum Report, APN 4453-005-018 (Vera), Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, December 27, 2007, Geotechnical Engineering Addendum Report, APN 4453-005-037 (Lunch), Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, December 28, 2007, Geotechnical Engineering Addendum Report, APN 4453-005-091 (Morleigh), Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, December 28, 2007, Geotechnical Engineering Addendum Report, APN 4453-005-092 (Mulryan), Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, July 14, 2008, Addendum Geotechnical Engineering Report #2, Response to the County of Los Angeles Department of Public Works, Geotechnical and Material Engineering Division, Soils Engineering Review Sheet Miscellaneous Application No 0706150005.

CalWest Geotechnical Engineering Consultants, July 22, 2008, Addendum Geotechnical Engineering Report #2, Response to the County of Los Angeles Department of Public Works, Geotechnical and Material Engineering Division, Soils Engineering Review Sheet Miscellaneous Application No 0706150004.

CalWest Geotechnical Engineering Consultants, July 23, 2008, Addendum Geotechnical Engineering Report #2, Response to the County of Los Angeles Department of Public Works, Geotechnical and Material Engineering Division, Soils Engineering Review Sheet Miscellaneous Application No 0706150004.

CalWest Geotechnical Engineering Consultants, May 1, 2009, Geotechnical Sections and Geologic Map, APN 4453-005-018.

CalWest Geotechnical Engineering Consultants, May 15, 2009, Geotechnical Engineering Supplemental Report, Proposed Compacted “Non-Structural” Fill Areas (Mulryan).

CalWest Geotechnical Engineering Consultants, July 7, 2009, Geotechnical Engineering Letter II.

CalWest Geotechnical Engineering Consultants, July 28, 2009, Geotechnical Engineering Letter, Preliminary Grading Plan Review, Proposed Single-Family Residential Development, Malibu Area, County of Los Angeles.

CalWest Geotechnical Engineering Consultants, May 3, 2010, Supplemental Geotechnical Engineering Letter #1, Additional Clarification of Design Recommendations and Response to California Coastal Commission Review Prepared by Cotton, Shires and Associates, Inc., Proposed Extension of Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, September 13, 2010, Supplemental Geotechnical Engineering Letter #2, Clarification to E-Mail From David Schrier (dschrier@cottonshires.com) Sent Friday, September 10, 2010 5:54 PM on Behalf of The California Coastal Commission, Proposed Extension of Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, September 30, 2010, Supplemental Geotechnical Engineering Letter #3, Additional Comments, Clarification and Response to Items Discussed at the Meeting Held at The California Coastal Commission on September 15, 2010; Proposed Extension of Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, October 13, 2010, Addendum to Supplemental Geotechnical Engineering Letter #3 dated September 30,2010, Additional Comments, Clarification and Response to Items Discussed at the Meeting Held at The California Coastal Commission on September 15, 2010; Proposed Extension of Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, November 1, 2010, Supplemental Geotechnical Engineering Letter #4, Response to Items Discussed Within the Memorandum Prepared by Cotton, Shires and Associates, Dated October 26, 2010 (included in Appendix A), Proposed Extension of Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, November 8, 2010, Supplemental Geotechnical Engineering Letter #5, Response to Discussion Items at The California Coastal Commission Meeting in San Francisco on November 2, 2010 Regarding Sweetwater Mesa Road Extension, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, November 11, 2010, Supplemental Geotechnical Engineering Letter #6, Proposed Staging Area, Compacted “Non-Structural” Fill, Sweetwater Mesa Road Extension, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, November 15, 2010, Supplemental Geotechnical Engineering Letter #7, Clarification of Design Loads for the Sweetwater Mesa Road Extension, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, December 20, 2010, RE: Draft report by Cotton Shires & Associates, Inc. dated December 17, 2010.

CalWest Geotechnical Engineering Consultants, December 27, 2010, Supplemental Geotechnical Engineering Letter #8, Additional Comments and Clarification of Stability Analysis and Geotechnical Design Load Criteria, Sweetwater Mesa Road Extension, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, January 17, 2011, Supplemental Geotechnical Engineering Letter #9, Additional Comments and Clarification of Stability Analysis and Geotechnical Design Load Criteria, Sweetwater Mesa Road Extension, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, January 20, 2011, Supplemental Geotechnical Engineering Letter #8 (Revised), Additional Comments and Clarification of Stability Analysis and Geotechnical Design Load Criteria, Sweetwater Mesa Road Extension, Malibu Area, County of Los Angeles, California.

Czerniak, E. 1957. "Resistance to Overturning of Single, Short Piles, in the Journal of the Structural Division, Proceedings of the American Society of Civil Engineers, Paper 1188, 1188-1 – 1188-25.

Cotton, Shires and Associates, Inc., March 8, 2010, Summary of Findings – Civil and Geotechnical Engineering and Engineering Geologic Peer Review Services, Sweetwater Mesa Development Project, Malibu, California.

Cotton, Shires and Associates, Inc., January 21, 2011, January 2011 Summary of Findings – Civil and Geotechnical Engineering and Engineering Geologic Peer Review Services, Sweetwater Mesa Development Project, Malibu, California.

County of Los Angeles, Dept of Public Works, Geotechnical and Materials Engineering Division, October 27, 2008, Soils Engineering Review Sheet, Review of Conceptual Design Pad for Single Family Residence and Access Road.

Hohbach-Lewin, Inc. Structural Engineers, December 6, 2010, Memo: Sweetwater Mesa Development Project – Civil and Geotechnical Engineering and Engineering Geological Peer Review.

Hohbach-Lewin, Inc. Structural Engineers, January 10, 2011, Memo: Sweetwater Mesa Road Extension Subject: Supplemental Geotechnical Letter #8, Additional comments and clarifications of Stability Analysis and Geotechnical Design Load Criteria.

Kane Geotechnical, October 15, 2007, Sweetwater Mesa Rockfall and Mitigation Study, Los Angeles County.

LC Engineering Group, Inc., September 27, 2009, Engineering Comments on California Coastal Commission's Draft of Scope of Work for Third Party Review, Sweetwater Mesa Development Project.

LC Engineering Group, Inc., October 20, 2009, Structural Analysis and Design: Sweetwater Mesa Rd (Sta 26+70 to 75+52.43), 2930 Sweetwater Mesa Road, Parts 1 and 2.

LC Engineering Group, Inc., January 27, 2010, Structural Analysis and Design: Sweetwater Mesa Rd (Sta 26+70 to 75+52.43), 2930 Sweetwater Mesa Road.

LC Engineering Group, Inc., May 3, 2010, Structural Analysis and Design: Sweetwater Mesa Rd (Sta 26+70 to 75+52.43), 2930 Sweetwater Mesa Road.

LC Engineering Group, Inc., May 28, 2010, Structural Analysis and Design: Sweetwater Mesa Rd (Sta 26+70 to 75+52.43), 2930 Sweetwater Mesa Road.

LC Engineering Group, Inc., November 16, 2010, Mesa Road Improvements from Sta: 26+70 to 75+53.34, Malibu, Los Angeles County, California (Sheets S-T to S-8).

Mountain Geology, Inc., September 26, 2006, Report of Limited Engineering Geologic Study, Proposed Water Main, Costa del Sol Way to APN 4453-005-038, -091, -037, -092, and -018, County of Los Angeles, California.

Mountain Geology, Inc., May 11, 2007, Report of Engineering Geologic Study – Proposed Custom Single-Family Residential Development (APN 4453-005-092, Mulryan).

Mountain Geology, Inc., May 11, 2007, Report of Engineering Geologic Study – Proposed Custom Single-Family Residential Development (APN 4453-005-091, Morleigh).
Mountain Geology, Inc., May 11, 2007, Report of Engineering Geologic Study – Proposed Custom Single-Family Residential Development (APN 4453-005-018, Vera), Electronic Copy.
Mountain Geology, Inc., May 11, 2007, Report of Engineering Geologic Study – Proposed Custom Single-Family Residential Development (APN 4453-005-037, Lunch), Electronic Copy.
Mountain Geology, Inc., August 28, 2007, Report of Engineering Geologic Study – Proposed Custom Single-Family Residential Development (APN 4453-005-038, Ronan).
Mountain Geology, Inc., December 18, 2007, Addendum Engineering Geologic Report #1 (APN 4453-005-037, Lunch).
Mountain Geology, Inc., December 19, 2007, Addendum Engineering Geologic Report #1 (APN 4453-005-092, Mulryan).
Mountain Geology, Inc., December 19, 2007, Addendum Engineering Geologic Report #1 (APN 4453-005-018, Vera).
Mountain Geology, Inc., December 20, 2007, Addendum Engineering Geologic Report #1 (APN 4453-005-091, Morleigh).
Mountain Geology, Inc., July 7, 2008, Addendum Engineering Geologic Report #2 (APN 4453-005-018, Vera) – Electronic Reference Copy.
Mountain Geology, Inc., July 8, 2008, Addendum Engineering Geologic Report #2 (APN 4453-005-091, Morleigh).
Mountain Geology, Inc., July 8, 2008, Addendum Engineering Geologic Report #2 (APN 4453-005-092, Mulryan).
Mountain Geology, Inc., July 8, 2008, Addendum Engineering Geologic Report #2 (APN 4453-005-037, Lunch).
Mountain Geology, Inc., May 18, 2009, Engineering Geologic Memorandum – Proposed Minor Modifications of Grading Plan, Northerly Terminus of Sweetwater Mesa Road.
Mountain Geology, Inc., April 23, 2010, Supplemental Engineering Geologic Report #1 – Engineering Geologic Responses to California Coastal Commission Engineering Geologic, Geotechnical Engineering and Civil Engineering Peer Review, APN 4453-005-037, -018, -038, -092, -091 Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.
Mountain Geology, Inc., September 14, 2010, Supplemental Engineering Geologic Report #2 – Engineering Geologic Responses to Email from David Schrier and Pat Shires Received on September 10, 2010, APN 4453-005-037, -018, -038, -092, -091 Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.
Mountain Geology, Inc., September 30, 2010, Supplemental Engineering Geologic Report #3 – Additional Responses to California Coastal Commission Engineering Geologic, Geotechnical Engineering and Civil Engineering Peer Review, APN 4453-005-037, -018, -038, -092, -091 Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.
Mountain Geology, Inc., October 29, 2010, Supplemental Engineering Geologic Report #4 – Additional Responses to California Coastal Commission Engineering Geologic, Geotechnical Engineering and Civil Engineering Peer Review, APN 4453-005-037, -018, -038, -092, -091 Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

Southern California Earthquake Center, June 2002, Recommended Procedures for Implementation of DMG Special Publication 117 for Analyzing and Mitigating Landslide Hazards in California.

Whitson Engineering, January 1, 2008, Revised March 9, 2009, 20' Driveway to Proposed Single Family Residence Plans, Sweetwater Mesa Road, (APN 4453-005-018, Vera).

Whitson Engineering, March 11, 2009, Driveway, Grading and Drainage Plans for a Single Family Residence (APN 4453-005-092, Mulryan).

Whitson Engineering, March 25, 2009, Driveway, Grading and Drainage Plans for a Single Family Residence (APN 4453-005-091, Morleigh).

Whitson Engineering, April 3, 2009, Driveway, Grading and Drainage Plans for a Single-Family Residence (CDP Submittal Not for Construction), (APN 4453-005-037, Lunch).

Whitson Engineering, April 28, 2009, Contour Grading Exhibit – 2839 Sweetwater Mesa Road (APN 4453-005-037).

Whitson Engineering, August 5, 2009, Driveway, Grading and Drainage Plans for a Single-Family Residence (CDP Submittal Not for Construction), (APN 4453-005-037, Lunch).

Whitson Engineering, August 5, 2009, 2851 U Sweetwater Mesa Road: Driveway, Grading and Drainage Plans for a Single-Family Residence (APN 4453-005-091, Morleigh).

Whitson Engineering, August 5, 2009, 2857 U Sweetwater Mesa Road: Driveway, Grading and Drainage Plans for a Single-Family Residence (APN 4453-005-092, Mulryan).

Whitson Engineering, August 5, 2009, 2863 U Sweetwater Mesa Road: Driveway, Grading and Drainage Plans for a Single-Family Residence (APN 4453-005-018, Vera).

Whitson Engineering, October 20, 2009, Sweetwater Mesa Project Summary Analysis Letter, Attn: Leslie Ewing of California Coastal Commission.

Whitson Engineering, October 21, 2009, Sweetwater Mesa Road Improvement Plans from Sta: 26+70 to 75+53.43.

Whitson Engineering, November 4, 2009 (Revised), Sweetwater Mesa Road Improvement Plans from Sta: 26+70 to 75+53.43.

Whitson Engineering, May 28, 2010, Sweetwater Mesa Road Improvement Plans from Sta: 26+70 to 75+53.43 (Site Plans)

Whitson Engineering, June 2, 2010, Sweetwater Mesa Road Improvement Plans from Sta: 26+70 to 75+53.43 (LACFD/CDP Submittal; Not for Construction).

Whitson Engineering, November 16, 2010 (revised), Plan Set, Sweetwater Mesa Road Improvements from STA: 26+70 to 75+53.43, Malibu, Los Angeles County, California.

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

January 21, 2010
P5020
By Email (lewing@coastal.ca.gov)

Ms. Lesley Ewing
Senior Coastal Engineer
CALIFORNIA COASTAL COMMISSION
45 Fremont Street, Suite 2000
San Francisco, California 94105-2219

SUBJECT: **Proposal for Civil and Geotechnical Engineering and Engineering Geological Peer Review Services**
RE: Sweetwater Mesa Development Project
Malibu, California

Dear Ms. Ewing:

Cotton, Shires and Associates, Inc. (CSA) is pleased to provide the California Coastal Commission (CCC) with this proposal for civil and geotechnical engineering and engineering geological peer review services in support of the CCC's review and analysis of the application for Coastal Development Permits 4-09-056 through 4-09-061. The project, as we understand it, consists of developing five residential lots along with a subdivision access road that would extend Sweetwater Mesa Road approximately one mile to the north of its present termination.

The new access road will be partially in the City of Malibu, but mostly in the County of Los Angeles, California. It is our understanding that the CCC is interested in having CSA review the engineering plans, geologic, engineering geologic, geotechnical, and supplemental reports for adequacy and compliance with the California Coastal Act policies that require the following: 1) new development in areas of high geologic, flood or fire hazard to be designed in such a way as to minimize risks to life and property; 2) new development must be designed to assure stability and structural integrity; and 3) new development shall consider scenic and visual qualities, protect views along the ocean and scenic coastal areas, minimize the alteration of natural landforms, be visually compatible with the character of surrounding areas, and, where feasible, to restore and enhance visual quality in visually degraded areas.

The project-specific requirements include stability review of the portion of the main 5-lot access road that is located within the County of Los Angeles, the individual access roads to the five residential lots, the water line extension to the five properties, and each of the five development areas. The peer review work will culminate in a written report summarizing findings and providing discussions and recommendations that will specifically address the following:

1) Evaluate whether the provided material is adequate to determine the aforementioned stability issues and if not, what additional material should be provided;

2) Assess whether the proposed remediation measures are adequate to provide stability for both static and dynamic loading conditions;

3) Assess whether the structural design (including pile diameters, spacing, embedment, steel reinforcement and orientation, force application, conformity to standards of practice, and ability to adequately resist lateral loads) of proposed remediation structures are appropriate for their intended purposes;

4) Assess whether the proposed remediation measures will potentially adversely impact slope stability;

5) Assess the necessity of fill proposed to be placed between Station 44+60 and Station 52+80 for stability purposes, fire department access and staging, and to evaluate the volume of fill being placed to eliminate off-haul;

6) Assess the compatibility and appropriateness of each stabilizing structure/improvement (cuts, fills, retaining walls, drainage, interconnecting piles, and cylindrical piles) necessary for the construction of the 5-lot access road;

7) Estimate the extent of additional disturbed areas and volumes of cut and fill necessary if the 1.5:1 slopes must be modified to 2:1;

8) Evaluate possible repairs to the pile supported roadway section in the unlikely event of failure due to landslide movement;

9) Assess the potential consequences of an unlikely failure of the pile supported roadway section;

10) Assess the potential failure mechanisms and repair options of the elevated roadway sections;

11) Confirm that roadway grade does not exceed the indicated 18.95 percent, and discuss issues associated with roadways constructed at this inclination;

12) Conduct a thorough spot-checking of calculated quantities for the following using provided topographic information:

    a) Volume and area of proposed cuts and fills [1.5:1 (H:V) slopes] of the roadway, residential access roadways, and building pad;

b) Volume and area of cuts and fills inclined at 2:1 (H:V) instead of 1.5:1 for the roadway, residential access roadways, and building pads;

c) Lengths and heights of retaining walls (roadway, residential access roadways, and building pads);

d) Length of roadway to be stabilized by slab piles and cylinder piles; and

e) Length and height of elevated roadway sections;

13) Evaluate the vulnerability of the roadway to geologic hazards;

14) Assess the constructability of the proposed roadway, residential access roadways and building pads;

15) Assess the long term effectiveness and appropriateness of the proposed stabilization elements; and

16) Identify conceptual level alternative designs and stabilization measures that would reduce grading and wall heights.

Upon the request of the applicant, and with concurrence with the CCC staff, we will participate in up to two meetings with the applicant's consultants and CCC staff. We will only participate in such meetings when CCC staff are present. County of Los Angeles staff will be invited and may attend these meetings at their discretion.

We further understand that the CCC needs the work completed in a timely manner so that they can prepare for a hearing on this matter to take place as soon as April of 2010. Assuming that authorization to proceed is granted in a reasonable time frame, a written report of our findings and recommendations would be needed on or before March 1, 2010. This report should provide detailed responses to the issues outlined above sufficient to develop evidence-based findings and with an explanation of how the above determinations and recommendations were made. Ongoing consultation with CCC staff and attendance and testimony at the CCC hearing for this project would also be required.

Consequently, we propose the following Scope of Work, Schedule, and Fee to perform the subject civil, geotechnical engineering and engineering geology peer review services.

SCOPE OF WORK

I. Initial Civil and Geotechnical Engineering and Geologic Evaluation

A. Evaluation of Aerial Photographs – Historical and relatively current aerial photographs will be obtained and analyzed with respect to slope stability considerations.

B. Review of Available Data - Published maps and site specific documents pertaining to the project and provided to us by the CCC, including reports, letters, memos and calculations, will be reviewed by engineering geologists, civil engineers, geotechnical engineers and structural engineers. We also request that the applicant's consultants provide us with the electronic versions of the drawings in AutoCAD-compatible format to assist us with our review. We will use AutoCAD Land Desktop and AutoCAD 2007 software to check quantities, etc.

C. Site Reconnaissances - Surficial inspections will be completed of the site and vicinity by an engineer and an engineering geologist and existing site conditions will be noted to formulate a preliminary understanding of the proposed project environment. Inspections of site earth materials and slopes will also be conducted, including preliminary engineering geologic mapping of site conditions using provided topographic base maps.

II. Engineering Geologic, Geotechnical and Civil/Structural Engineering Assessment and Evaluation of Site Conditions

A. Preliminary Assessments and Evaluations – Based on our review of the site conditions, aerial photographs, published maps and site specific documents (including electronic files of the drawings) provided to us, we will develop preliminary assessments and evaluations to address the CCC's questions, concerns and requests regarding the construction of the proposed roadway, residential access roadways and building pads.

III. Consultation, Reporting and Meetings

A. Consultation and Reporting – We will consult with CCC staff on a regular ongoing basis and we will prepare a peer review letter report which will contain our assessments and evaluations of the site conditions, reviewed documents, and address each of the CCC's above listed questions, concerns and requests. CCC staff will be included in all telephone/meeting contacts.

B. Meetings – In addition to the site inspection, we have budgeted for attending two meetings with CCC staff and the applicant's consultants at our Los Gatos office or during the field trip.

C. Coastal Commission Hearing - We have budgeted for preparing a PowerPoint presentation to summarize our peer review assessments and evaluations, and for attending a CCC hearing to present our findings.

SCHEDULE

Upon our receipt of a signed agreement, we will begin our peer review of provided documents and our evaluation of the site. At this time, we anticipate that the evaluation will take approximately five to six weeks to complete. Assuming timely authorization, we will endeavor to meet the CCC's expectation that the peer review report be provided by March 1, 2010, and we will be available to participate in the CCC hearing as early as April of this year.

FEE

We propose to invoice you for our services on a time-and-expenses basis in accordance with the attached Schedule of Charges. We estimate that our fees and expenses for Tasks I through III outlined above will be:

Task	Estimated Cost Range
I. – Initial Evaluation	**$24,000 to $26,000**
A. Evaluation of Aerial Photos - $1,500 to $2,000	
B. Review Available Data - $18,000 to $19,000	
C. Site Reconnaissances - $4,500 to $5,000	
Senior Principal Engineer (32 hours x $250 = $8,000)	
Principal Engineer/Geologist (32 hours x $210 = $6,720)	
Senior Staff Engineer/Geologist (40 hours x $130 = $5,200)	
Supervising Structural Engineer (20 hours x $175 = $3,500)	
Travel Cost, Mileage, Aerial Photographs ($1,750)	
II. – Assessment and Evaluation of Site Conditions	**$19,000 to $20,500**
Senior Principal Engineer (16 hours x $250 = $4,000)	
Principal Engineer/Geologist (20 hours x $210 = $4,200)	
Senior Staff Engineer/Geologist (64 hours x $130 = $8,320)	
Supervising Structural Engineer (20 hours x $175 = $3,500)	
III. – Consulting, Reporting and Meetings	**$27,500 to $30,000**
A. Consultation and Reporting - $14,000 to $14,500	
B. Meetings - $4,500 to $5,500	
C. Coastal Commission Hearing - $9,000 to $10,000	
Senior Principal Engineer (50 hours x $250 = $12,500)	
Principal Engineer/Geologist (50 hours x $210 = $10,500)	
Senior Staff Engineer/Geologist (16 hours x $130 = $2,080)	
Technical Illustrator (40 hours x $85 = $3,400)	
Reproduction Costs and Admin/Accounting ($1,250)	
(If meetings are held at CCC's office in S.F., add $1,500 per meeting)	

We will invoice the CCC monthly on a time and expenses basis for Tasks I through III for an amount ranging from $70,500 to $76,500 and will not exceed $76,500 without prior written authorization. Attendance at additional meetings or hearings (beyond the budgeted two meetings and one hearing) will be billed on a time-and-expense basis in accordance with our attached Schedule of Charges. If meetings are held in San Francisco, please add $1,500 per meeting to the not-to-exceed amount.

AGREEMENT

If you agree with the Scope of Work, Schedule, and Fee outlined above, as well as the attached Schedule of Charges, Limitations, and Terms, please sign one copy of this proposal and return it to our office or incorporate it as an exhibit into a contract. Receipt of the signed proposal or contract will constitute authorization for us to proceed.

We look forward to providing you with the professional services discussed above. If you have any questions, or need additional information, please contact us.

Very truly yours,

COTTON, SHIRES AND ASSOCIATES, INC.

Patrick O. Shires
President and Senior Principal Geotechnical Engineer, GE 770

REGISTERED PROFESSIONAL ENGINEER
PATRICK O. SHIRES
No. 770
Exp. 3/31/10
GEOTECHNICAL
STATE OF CALIFORNIA

Approved and Authorized By

Date

POS:DTS:TS:MP:st

Attachment: CSA Schedule of Charges, Limitations and Terms

CALIFORNIA COASTAL COMMISSION
45 FREMONT, SUITE 2000
SAN FRANCISCO, CA 94105-2219
VOICE AND TDD (415) 904-5200
FAX (415) 904-5400

February 12, 2010

FROM: Lesley Ewing

TO: Patrick Shires, Cotton Shires and Assoc.
Dennis Long, Monterey Bay Sanctuary Foundation
Ted Harris, California Strategies

SUBJECT: ADDENDUM to Cotton Shires and Assoc. Sweetwater Mesa Proposal 1.21.2010

This memo is to explicitly state that the scope of work for your third party review, as specified in Item 16 of the 21 January 2010 Sweetwater Mesa Proposal does not extent to examination of alternative building sites. As such, Item #16 should be modified to read:

"Identify conceptual level alternative designs and stabilization measures that would reduce grading and wall heights *The peer review does not include review or identification of alternative home site locations*."

The amended language is provided in underlined italics.

No other changes or modifications to the Scope of Work are proposed and all items in the scope of work shall be undertaken as previously specified.

March 8, 2010
E5050
By email (lewing@coastal.ca.gov)

Ms. Lesley Ewing
Senior Coastal Engineer
CALIFORNIA COASTAL COMMISSION
45 Fremont Street, Suite 2000
San Francisco, California 94105-2219

SUBJECT: **Summary of Findings - Civil and Geotechnical Engineering and Engineering Geologic Peer Review Services**
RE: Sweetwater Mesa Development Project
Malibu, California

Dear Ms. Ewing:

Cotton, Shires and Associates, Inc. (CSA) is pleased to provide the California Coastal Commission (CCC) with this summary of our findings in regard to the civil and geotechnical engineering and engineering geologic peer review services we provided in support of the CCC's review and analysis of the application for Coastal Development Permits 4-09-056 through 4-09-061 for the Sweetwater Mesa Development Project in Malibu, California. The project consists of developing five residential lots along with a subdivision access road that would extend Sweetwater Mesa Road approximately one mile to the north of its present termination.

The new access road is proposed partially in the City of Malibu, but mostly in the County of Los Angeles, California. It is our understanding that our task was to review the engineering plans and calculations, geologic, engineering geologic, geotechnical, and supplemental reports pertaining to the portion of project within the County of Los Angeles for adequacy and compliance with the California Coastal Act policies that require the following: 1) new development in areas of high geologic, flood or fire hazard to be designed in such a way as to minimize risks to life and property; 2) new development must be designed to assure stability and structural integrity; and 3) new development shall consider scenic and visual qualities, protect views along the ocean and scenic coastal areas, minimize the alteration of natural landforms, be visually compatible with the character of surrounding areas, and, where feasible, to restore and enhance visual quality in visually degraded areas.

The project-specific requirements include stability review of the portion of the main 5-lot access road that is located within the County of Los Angeles, the individual access roads to the five residential lots, the water line extension to the five properties, and each of the five development areas. The peer review work has culminated in this written report summarizing our findings and providing discussions and recommendations.

SCOPE OF WORK

I. Initial Civil and Geotechnical Engineering and Geologic Evaluation

A. Evaluation of Aerial Photographs – Historical and relatively current aerial photographs were obtained and analyzed with respect to slope stability considerations.

B. Review of Available Data - Published maps and site specific documents pertaining to the project and provided to us by the CCC, including reports, letters, memos and calculations, were reviewed by engineering geologists, civil engineers, geotechnical engineers and structural engineers. The applicant's consultants also provided us with the electronic versions of the drawings in AutoCAD-compatible format to assist us with our review. We used AutoCAD Land Desktop and AutoCAD software to check quantities, etc.

C. Site Reconnaissance - Surficial inspections were completed of the site and vicinity by an engineer and engineering geologists and existing site conditions were noted to formulate a preliminary understanding of the proposed project environment. Inspections of site earth materials and slopes were also conducted, including preliminary engineering geologic mapping of site conditions using provided topographic base maps.

II. Engineering Geologic, Geotechnical and Civil/Structural Engineering Assessment and Evaluation of Site Conditions

A. Preliminary Assessments and Evaluations – Based on our review of the site conditions, aerial photographs, published maps and site specific documents (including electronic files of the drawings) provided to us, we developed preliminary assessments and evaluations to address the CCC's questions, concerns and requests regarding the construction of the proposed roadway, residential access roadways and building pads.

III. Consultation, Reporting and Meetings

A. Consultation and Reporting – We consulted with CCC staff on a regular ongoing basis and we prepared this peer review letter-report containing our assessments and evaluations of the site conditions, reviewed documents, and addressed each of the CCC's questions, concerns and requests. CCC staff was included in all telephone/meeting/email contacts.

B. Meetings – In addition to the site inspection, we attended two meetings with CCC staff at our Los Gatos office and one meeting with the applicant's consultants during the initial field trip.

C. Coastal Commission Hearing - We will prepare a PowerPoint presentation to summarize our peer review assessments and evaluations, and attend a CCC hearing to present our findings.

FINDINGS, CONCLUSIONS AND RECOMMENDATIONS

Engineering Geologic Evaluation Introduction – To provide a basis upon which to review the geotechnical and engineering aspects of the proposed development, we performed an engineering geologic evaluation of the project. This evaluation included review of historical stereo-pair aerial photographs (1929, 1952, 1993, and 2000) and historical oblique aerial photographs (1993, 2008, and 2009), and performance of limited engineering geologic field mapping. Our evaluation also included review of the submitted geologic reports, geologic maps, geologic cross sections, and exploratory borehole and trench logs by the Project Engineering Geologist, Mountain Geology, Inc. (MGI). The fundamental role of the engineering geologist is to first recognize the primary geologic hazards with the potential to impact the proposed development, and second to characterize these geologic hazards so that appropriate geotechnical engineering analyses can be performed. In this section of our letter-report, we summarize our evaluation of the engineering geologist's recognition and characterization of the site geologic conditions and geologic hazards.

Geologic Hazard Recognition – MGI has recognized that landsliding, seismic shaking, rockfalls, and bedrock shattering have the potential to adversely impact the proposed development. MGI has stated that landslide debris underlies the majority of the subject property, and has recommended that mitigation measures be implemented to provide the appropriate required factor of safety for the proposed access road and residences. CSA is in agreement that the majority of the site is underlain by landslide debris, which in general, has been shed westward from the prominent north-south trending ridgeline. Three of the proposed residential structures are located atop the prominent ridgeline on bedrock materials of the Vaqueros Formation (see Figure 1, Aerial Site View). Our review of the proposed development reveals that the Lunch residence is the only living space to be constructed atop landslide debris, as mapped by MGI; however, it is our opinion that a large portion of the Morleigh property, including the residence site, may also be underlain by a large landslide.

CSA's review of aerial photographs reveals the likely presence of four large landslides along the western flank of the ridgeline (which we will refer to as landslides 1 through 4 in the following text), with Landslide 1 on the Vera property (and property to the south of Vera), Landslide 2, a large, mostly evacuated landslide on the Mulryan and Lunch properties, Landslide 3 on the Morleigh property, and Landslide 4 north of the Morleigh property (see Figure 2, CSA Photo-Interpretive Landslides). MGI has mapped Landslides 1, 2 and 4, but does not map the Morleigh site as a landslide (CSA potential Landslide 3). The distribution of landslides as depicted on the MGI Geologic Map is very general in nature, and thus, it is difficult to differentiate large from small landslides, but in general, with the exception of CSA's Landslide 3, MGI appears to have recognized the majority of the landslides within the project area. In addition, MGI appears to have adequately recognized other geologic hazards with the potential to adversely impact the proposed site development.

In order to illustrate our peer review interpretation of the site geology based on our review of aerial photographs, limited field mapping and review of the MGI mapping and subsurface information, we prepared the attached Plate 1 – Peer Review Engineering Geologic Map.

Geologic Characterization – MGI has performed geologic field mapping, evaluated aerial photographs, performed subsurface exploration, and developed geologic cross sections to portray the site geologic conditions. CSA has reviewed the geologic maps, cross sections, and borehole data submitted by MGI, and has provided the following assessment of the site geologic characterization:

Geologic Mapping: On the MGI Geologic Map, MGI has identified landslide areas using gray shading and identified non-landslide areas as white. Within the gray landslide areas, MGI does not, in general, differentiate the various types of landslides or slope movements at the site (i.e., shallow landslide, deep landslide, slope wash, talus, etc.), nor do they differentiate the different parts of each landslide (i.e., headscarp, toe, lateral margin, internal slide, etc.). Additionally, the movement directions of the various landslides are not well constrained. It is our opinion that, for certain portions of the proposed development, this results in overly conservative design assumptions. One example of this is near the first switchback north of the proposed Lunch residence where in exploratory boring B-9, approximately 52 feet of landslide debris was encountered. This boring appears to be located along the northern margin of the large, mostly evacuated Landslide 2, which appears to have moved nearly due west (see Plate 1 – Peer Review Engineering Geologic Map). The slope above this boring is part of the old headscarp/sidescarp of the large landslide, and appears to be mantled with shallow slopewash, but should not have a basal shear surface contiguous with the large landslide (as shown on MGI Section E-E'). In boring B-9, MGI encountered approximately 12 feet of slopewash, which as one moves northward off the large landslide and onto the scarp, should thin upslope. Thus, planned mitigation elements along this section could be over-designed. We recommend that a well-placed boring along the road alignment above B-9 could provide valuable subsurface information to help refine, and hopefully reduce, the mitigation design.

Similar refinement of the landslide type and distribution may also have positive implications for Landslide 1, near Cross Section U-U' and V-V', where the obvious headscarp of the large landslide contains surficial slopewash and talus (see Plate 1 – Peer Review Engineering Geologic Map), and the basal shear surface of the large landslide should not extend upslope as shown on MGI's U-U' and V-V'.

Mapping Recommendations – We recommend that MGI incorporate landslide geomorphology into their geologic mapping to help refine the shape, depth interpretations, and direction of the landslides at the site.

Additional field mapping should be considered in the vicinity of CSA Landslide 3. The aerial photographs appear to show a pronounced headscarp graben with south-directed drainage leaving the graben area. Consideration should be given to mapping the toe region of the slope, and documenting the topographic conditions of the main body of the slope. Our field mapping revealed highly irregular topography in the main portion of the potential landslide, and the Cross Section M-M' surface profile through this slope is suggestive of landslide geomorphology. Additionally, projection of borehole information from borings B-17 and B-19 onto this cross section is suggestive of landslide shearing in this area.

Subsurface Exploration: A total of 36 large-diameter boreholes, 1 small-diameter borehole, and 16 backhoe test pits were excavated by MGI to explore the subsurface conditions at the site. The large-diameter boreholes were drilled to a maximum depth of 70 feet. A total of 14 large-diameter boreholes were drilled within landslide debris, with the basal shear surface of the landslide identified by MGI in the boreholes; however, MGI did not identify striations (and their orientations) on any of the basal shear surfaces. All large-diameter borehole exploration performed within landslides was conducted near the head of these landslides. In general, the descriptions of the landslide basal shear surfaces are short, and are not unique from many other borehole material descriptions. The logs of the exploratory boreholes do not indicate that hand sampling was performed on any of the basal shear surfaces.

Subsurface Exploration Recommendations –

- We recommend that additional subsurface exploration be performed along the roadway north of B-9 to characterize the subsurface materials along the steep slope.

- We recommend that exploratory trenching be considered in the gently sloping area (possible graben) near the proposed Morleigh residence to help determine the presence or absence of landsliding. Additional borings may also be necessary to help constrain the geologic conditions in this area.

- Additional subsurface exploration should be performed downslope of the proposed Lunch residential site. Exploratory borings were performed in the vicinity of the proposed residence, but no exploration was performed downslope in the vicinity of the roadway. There are no other positive picks on the basal shear surface for this landslide, thus, there is nothing to constrain the location of the slide plane in the vicinity of the roadway where mitigation elements are to be implemented (see MGI Cross Section G-G').

- We recommend that additional borehole exploration be performed with the intent of obtaining hand samples of the slide plane materials for appropriate laboratory testing, and to further constrain the subsurface landslide geometries where only one positive pick on the basal shear surface has been obtained.

Geologic Cross Sections: Many of the geologic cross sections could be improved with more refined geologic and geomorphic landslide mapping and subsurface exploration. Along some of the cross sections, this re-interpretation may result in shallower mitigation elements, such as along Cross Sections B-B', E-E', F-F', U-U', and V-V'. Other geologic cross sections lack sufficient subsurface data to accurately interpret the subsurface conditions (i.e., G-G', K-K'). Since nearly all of the subsurface geologic data are from near the head of the landslides, the presented basal shear surface geometry is not likely accurate. Therefore, any back-calculated strengths, or forward analyses based upon these geometries are likely inappropriate. Several of the cross sections (such as U-U' and V-V') do not appear to have incorporated field data appropriately to the cross sections. In these areas, very steep basal shear surfaces (47 to 54 degrees) were documented in the boreholes, but the basal shear surface shown on the sections is much shallower. There may have been an attempt by MGI to depict the apparent dip of these features, thus the lower angle; however, this appears to present an inaccurate depiction of the actual site conditions.

Cross Section Recommendations **-** The geologic cross sections developed by MGI should be updated to more accurately reflect the landslide geomorphology at the site.

The geologic cross sections should be aligned to the most critical portion of the landslides parallel with the estimated movement direction.

Summary of Geologic Characterization – MGI has performed a geologic investigation where valuable surface and subsurface geologic information has been gathered, and specific geologic hazards have been identified. It is our opinion that the geologic characterization of the site to date is an excellent feasibility-level investigation that helps focus attention on specific areas that require further characterization. In our opinion, the geologic characterization to date does not appear to provide sufficient accuracy, detail, or aerial coverage for design level analyses.

Summary of Geotechnical Engineering Evaluation – During our review, we identified various aspects of the investigation, analysis and design that were not in conformance with typical investigations for a project of this magnitude and complexity. The following discussion summarizes some of these aspects, and provides a general recommendation for additional investigation (which can be combined with the additional engineering geologic investigation), laboratory testing and analysis that should be undertaken to better quantify key geotechnical design criteria parameters and landslide loading scenarios. Unfortunately, many of these points are repeated as part of our answering the Summary of Requested Scope of Work in an organized manner.

Geotechnical Subsurface Investigation – It appears that for the entire subsurface investigation program, only one small-diameter boring was utilized for this project. We further understand that this boring was a core-sample exploratory borehole. Consequently, it doesn't appear that undisturbed samples were available or used for laboratory testing. It appears that all of the samples used for laboratory testing were either disturbed samples obtained during downhole logging (also called "grab samples"), or were driven by the Kelly bar of the drill rig (although widespread in practice for sampling of large-diameter borings, this is not an ASTM-approved sampling method for relatively undisturbed samples). However, on many of the boring logs, there is not a description of the type or method of sampling. We recommend that a detailed description of the type and method of sampling be provided for all samples tested in the laboratory if not all samples collected during the subsurface investigation program.

Laboratory Testing – There doesn't appear to be a comprehensive discussion regarding the methodology of the laboratory testing, or an explanation regarding the laboratory test results, so some of our comments regarding the laboratory testing are based on inferences.

- Since it is not apparent how samples were obtained during the subsurface investigation, please explain how the Direct Shear (DS) test samples were obtained/prepared;

- Since it is not apparent how samples were obtained during the subsurface investigation, please explain how the unit weight values used in the slope stability analysis were determined;

- It appears that additional subsurface exploration should be considered to obtain relatively undisturbed samples of the landslide debris material for laboratory testing;

- A total of 38 Direct Shear (DS) tests were reported and described as: 1) "Along Bedding"; 2) "ULT" (ultimate?); 3) "RSHR" (residual shear?); and 4) "Remolded 90%". Please clarify how each sample of these four types of tests were prepared, and the intended future use during analysis;

- Of those 38 DS tests reported, three were described as Qls plane and three as Qls. Five of these tests were run as RSHR and the sixth one as ULT. Please clarify the intent and use (pertaining to slope stability analysis and design criteria) of these six tests;

- From laboratory test result tables, it appears that for the three DS tests on "Qls Plane" material, cohesions varied from 190 psf to 290 psf, and the ϕ varied from 24^0 to 31^0. The Consultant should justify that sufficient laboratory testing on landslide plane material has been accomplished or consider additional testing;

- From test result tables, it appears that for the three DS tests on "Qls" material, cohesions varied from 230 psf to 540 psf and the ϕ varied from 23^0 to 34^0. The Consultant should justify that sufficient laboratory testing on landslide debris material has been accomplished or consider additional testing;

- All of the slope stability analyses appear to be based on shear strength parameters from a DS test described as RSHR on Qls from boring B-12 at a depth of 28 feet. The Consultant should discuss the applicability of using only one residual shear strength for all of the site landslides, including landslide planes shearing along bedding and others shearing across bedding.

- The selected residual shear strength includes a significant cohesion component. Typically, cohesion is set to 0 for existing landslides/residual strength parameters. The Consultant should consider the use of 0 cohesion or at least minimal cohesion for the existing landslide basal shear strength or provide evidence supporting the use of significant cohesion in these slope stability analyses.

- Because of the inability to control drainage, the susceptibility to disturbance, the potential for minor disconformities skewing the results, and a forced direction of shearing in the thin sample, DS tests are generally considered less reliable for determining in situ shear strengths of landslides. It has been shown that Triaxial Compression tests (consolidated undrained) should provide more accurate shear strength parameters for the landslide debris material. For a project of this scope and complexity, it is our opinion that Triaxial Compression Consolidated Undrained tests on relatively undisturbed samples should be considered for the landslide debris material.

- Because of the difficulty in obtaining and shearing landslide plane material under in situ conditions, correlations with Atterberg limits and clay fractions have been shown to provide reasonable residual and fully-softened shear strength values. The Consultant has included a copy of the typical correlations between secant residual strength and liquid limits (in a response letter to the County of Los Angeles dated December 28, 2007). Unfortunately, only five Atterberg limits tests were performed for the entire project, and four of those were in Qls material while the fifth (with the highest Liquid Limit) was performed on siltstone bedrock. Additional samples should be obtained along basal landslide shear planes, and tested, including Atterberg limits and clay fraction testing, and torsional ring shear strength testing should be considered.

Slope Stability Analysis and Structural Design – Following completion of the recommended additional laboratory testing, slope stability analyses should be repeated incorporating updated engineering geologic modifications (described above) and recommendations to the slope stability analysis provided in the following sections. If Landslide 3 is found to be present, then laboratory testing on landslide-specific samples, appropriate analysis and structural design measures will be necessary to address it. In addition, the stabilizing structures should be reanalyzed and redesigned in accordance with responses to the recommendations presented in the following sections.

SUMMARY OF REQUESTED SCOPE OF WORK

The following are the 16 itemized requests of the CCC in the order requested (in *italics*), with the corresponding CSA response following the requested scope.

1) Requested Scope - Evaluate whether the provided material is adequate to determine the aforementioned stability issues and if not, what additional material should be provided.

1) Conclusions and Recommendations – Based on our evaluation of the provided materials, there are several items that should be provided in order to allow an adequate basis for determining the project stability and other issues. Many of these items are apparent in the text responding to the remaining requested scope items and in the text of our general evaluation summary of the geology, geotechnical engineering, civil engineering and structural engineering work. Some of these materials include:

a) Engineering geologic mapping of individual landslide parts (i.e., headscarp regions differentiated from unit surfaces, grabens, etc.);

b) Engineering geologic indication of direction of landsliding;

c) More detailed description of landslide shear planes on boring logs (thickness of shear zone, orientation of shear zone, indication of striae and polishing on shear surfaces, etc.);

d) Engineering geologic cross sections that are more representative of critical slope stability with respect to the proposed access road (i.e., oriented in the direction of landsliding and through the critical portions of the access road);

e) Subsurface exploration evidence (such as trenching) to prove or disprove the presence of landsliding in the vicinity of the Morleigh residence (CSA Landslide 3) with an apparent graben area (based on evaluation of aerial photographs) and currently not mapped as a landslide;

f) Detailed description of sampling procedure and whether samples tested were remolded, disturbed, moderately disturbed or undisturbed;

g) Detailed description of direct shear testing procedure and sample preparation if remolded (how appropriate unit weight and moisture content determined, etc.);

h) A more comprehensive laboratory test program should be undertaken based on the understanding that each landslide should be analyzed separately, and this should include obtaining relatively undisturbed samples and performing Triaxial Compression tests (TX/CU) on landslide debris from each separate landslide to be mitigated. Furthermore, grab samples should be obtained from the landslide basal shear plane of each landslide to be mitigated and Atterberg Limits tests performed on each grab sample to obtain correlations with residual shear strengths (Stark, et al., May 2005). According to the Southern California Earthquake Center, June 2002, Recommended Procedures for Implementation of DMG Special Publication 117 for Analyzing and Mitigating Landslide Hazards in California: "DS testing devices can be used to subject a sample to multiple cycles of shearing, which allows an estimation of residual strength. Unfortunately, the results may be unconservative … and should always be checked against either correlations … or results of ring shear testing …". Consideration should be given to torsional ring shear strength testing (fully softened and residual shear strength) of representative basal landslide shear plane materials;

i) Detailed description of the basis for the active soil pressure of 30 pcf equivalent fluid used in design;

j) Detailed description of the basis for 82% active load left over after seismic shaking;

k) Slope stability calculations using a more rigorous Factor of Safety procedure that satisfies both moment and force equilibriums;

l) Slope stability analysis of impact of fill prisms placed on landslides on slope stability; and

m) Grading plans for individual residence house sites indicating balancing of cut and fill or how much additional fill will be generated and where it will be placed.

2) *Requested Scope - Assess whether the proposed remediation measures are adequate to provide stability for both static and dynamic loading conditions;*

2) Conclusions and Recommendations - For design purposes, the geotechnical engineering consultant (CalWest Geotechnical, Inc., referred to herein as "CalWest") divided the proposed access road into 6 regions where the road crosses over landslides. The regions are labeled A through F and the approximate access road stations that define the limits of each region are provided in the following table.

Region Identification	Start Station	End Station
A	26+50	28+45
B	28+45	29+75
C	29+75	31+25
D	43+80	48+00
E	48+00	51+00
F	51+00	57+00

Structural design loads within each of the regions were based on either lateral loads from static and pseudo-static slope stability analyses or lateral active pressure loads from localized retaining wall analysis, whichever was greater. Design loads from two-dimensional cross sections were then averaged along the length of the road for each region. Details of the analyses within each region are provided below. It should be noted that at the terminus of Regions C, D and F, the structural piers do not extend to the northern limits the mapped landslides. Explanation for the reasoning behind this design decision should be provided.

2.1 Region A

The design load within Region A was determined by CalWest to be 130 kips/ft. This load was an approximate "average" of required loads from analyses of Cross Sections U-U', V1-V1', and V2-V2'. The slope stability analysis (SSA) of Cross Section U-U' was performed in May 2007, with preliminary material strength parameters. Although CalWest subsequently updated their material strength parameters and increased the strength of the landslide debris, this preliminary analysis was considered when determining the structural design loads within this region.

Based on our review of the analyses performed along Cross Sections U-U', V1-V1', and V2-V2', we have the following comments and observations:

- For purposes of SSA, Cross Section U-U' is a global cross section from which the entire length of the landslide mass is analyzed. Cross Sections V1-V1' and V2-V2' are local cross sections from which only the portion of the landslide mass upslope and underneath the proposed access road is analyzed;

- SSA was performed using the Simplified Bishop factor of safety (FOS) procedure. This procedure only satisfies moment equilibrium and is, therefore, less rigorous than FOS procedures which simultaneously satisfy both moment equilibrium and force equilibrium. Independent SSA performed by CSA indicate the required lateral loads calculated by CalWest using the Simplified Bishop FOS procedure along Cross Sections V1 and V2 result in safety factors which do not meet 1.5 compliance when evaluated with a FOS procedure which satisfies both moment and force equilibriums simultaneously. Based on our independent SSA, CSA estimates the required lateral loads to be roughly 44% and 59% higher for Cross Sections V1-V1' and V2-V2', respectively, when both force and moment equilibriums are satisfied. While the Simplified Bishop procedure is used in the industry, we recommend that, for a structural system of this magnitude, consideration be given to using a procedure which simultaneously satisfies both moment and force equilibriums;

- A surcharge load of 100 psf was applied by CalWest at the location of the proposed access road for the analysis of Cross Section U-U'. There were no surcharge loads applied for the CalWest analyses of Cross Sections V1-V1' and V2-V2'. The Consultant should re-analyze these cross sections using a surcharge load;

- The required lateral loads from SSA were then reduced to account for a "favorable active load" induced on the proposed piles by the adjacent landslide debris downslope. The favorable active load appears to have been considered over 82% of the pile height in accordance with a Newmark seismic displacement analysis. It is not clear to us what active earth pressure coefficient was applied or how the 82% was determined. The Consultant should clarify how this was done and the basis for this assumption; and

- The "average" design load for Region A was based on nonlinear interpolation between required loads at Cross Sections U-U′, V1-V1′ and V2-V2′. The Consultant should justify the procedure of averaging required loads to develop a design load, based on the fact that by definition a significant portion (50 percent if linear average) of the structure will be under-designed.

Based on the comments and observations provided above, for the engineering geologic model analyzed, **it appears that the design loads calculated for static and pseudo-static stabilization within Region A are inadequate.** We recommend that CalWest consider re-analyzing Cross Sections U, V1, and V2 using a rigorous FOS procedure (i.e., a procedure which simultaneously satisfies both moment and force equilibriums). If cross sections are reconstructed based on the engineering geologic discussions provided above, then the updated and modified cross sections should be utilized. It is also our opinion that Cross Section U should be updated with the most recent material strength parameters and geometrically truncated to be consistent with the localized analyses performed along Cross Sections V1-V1′ and V2-V2′. In addition, the "average" design loads appear to be lower than the calculated design loads at Cross Section U-U′ and V1-V1′, but much higher than the calculated design load at Cross Section V2-V2′. We are concerned that averaging the design loads in this manner could lead to parts of the road foundation which are overstressed and, consequently, lead to a progressive "unzipping" failure of the road foundation.

2.2 Region B

The design load within Region B was determined by CalWest to be 50.7 kips/ft. This load was determined using a procedure similar to the procedure applied within Region A. Within Region B, the calculated design loads along Cross Section V2-V2′ and V-V′ were considered.

Based on our review of the analyses performed along Cross Sections V2-V2′ and V-V′, we have the following comments and observations:

- For purposes of SSA, Cross Section V-V′ is a global cross section from which the entire length of the landslide mass is analyzed. As mentioned above, Cross Section V2-V2′ is a local cross section from which only the portion of the landslide mass upslope and underneath the proposed access road is analyzed;

- SSA was performed using the Simplified Bishop FOS procedure which only satisfies moment equilibrium. See above for recommendations regarding performing SSA using programs which simultaneously satisfy both moment equilibrium and force equilibriums;

- A surcharge load of 100 psf applied at the location of the proposed access road for the analysis of Cross Section V-V′. However, there was no surcharge load applied for the analysis of Cross Section V2-V2′. The Consultant should re-analyze this cross section using a surcharge load;

- It appears that the reported design load at Cross Section V-V′ was based on an equivalent fluid pressure (EFP) of 30 pcf. The Consultant should clarify how an EFP of 30 pcf was derived; and

- The strength parameters of the slide surface are inconsistent between Cross Sections V-V′ and V2-V2′. The Consultant should provide justification for inconsistencies is shear strength parameters for the same landslide.

Based on the comments and observations provided above, for the engineering geologic model analyzed, **it appears that the design loads calculated for static and pseudo-static stabilization within Region B are inadequate.** We recommend that CalWest re-analyze Cross Sections V2-V2′ and V-V′ using a rigorous FOS procedure. If cross sections are reconstructed based on the engineering geologic discussions provided above, then the updated and modified cross sections should be utilized. It is also our opinion that Cross Section V-V′ should be geometrically truncated to be consistent with the localized analyses performed along Cross Sections V1-V1′ and V2-V2′. Material strength parameters should be made consistent for each of the cross sections, when in the same landslide unless there is appropriate evidence presented to justify using differing parameters.

The use of an equivalent fluid pressure (EFP) of 30 pcf to represent earth pressures induced by a landslide should also be justified and an explanation provided as to how it was determined. In addition, the "average" design loads appear to be lower than the calculated design loads at Cross Section V2-V2′, but higher than the calculated design load at Cross Section V-V′. Again, we are concerned that averaging the design loads in this manner could lead to parts of the road foundation that are overstressed and, consequently, lead to a progressive "unzipping" failure of the road foundation.

2.3 Region C

The design load within Region C was determined by CalWest to be 17.6 kips/ft. This load was determined using a procedure similar to the procedure applied within Regions A and B. Within Region C, the calculated design loads along Cross Section V-V′ were considered. It was assumed that there would be a required design load of 0 kips/ft at STA31+25, which corresponds to the margin of the landslide.

Comments and observations related to the analysis of Cross Section V-V′ were presented in Section 2.2, above. Based on these comments and observations, for the engineering geologic model analyzed, **it appears that the design loads calculated for static and pseudo-static stabilization within Region C are inadequate.** We recommend that CalWest consider re-analyzing cross section V using a rigorous FOS procedure. If cross sections are reconstructed based on the engineering geologic discussions provided above, then the updated and modified cross sections should be utilized. As discussed in Section 2.1, it is our opinion that Cross Section V-V′ should be geometrically truncated to be consistent with the localized analyses performed along Cross Section V1-V1′ and V2-V2′. Material strength parameters should be made consistent for each of the cross sections or evidence should be presented which justifies the use of differing parameters.

Again, the "average" design loads within this region appear to be lower than the calculated design load at Cross Section V-V′. Again, we are concerned that averaging the design loads in this manner could lead to parts of the road foundation becoming overstressed and consequently lead to a progressive "unzipping" failure of the road foundation.

It appears that the mitigation structures (piles) don't extend to the margins of the landslides and stop short by about 30 feet. The Consultant should confirm that the mitigation structures are positioned accurately across the landslide to provide the anticipated support.

2.4 Region D

The design load within Region D was determined by CalWest to be 53.0 kips/ft. This load appears to have been determined by taking the arithmetic average of the required loads along Cross Sections G1-G1′ and G2-G2′. It appears that SSA of Cross Section G-G′ was also considered in the analysis of this region, but was not found to be critical for design purposes.

Based on our review of the analyses performed along Cross Sections G-G′, G1-G1′ and G2-G2′, we have the following comments and observations:

- For purposes of SSA, Cross Section G-G′ is a global cross section from which the entire length of the landslide mass is analyzed. Cross Sections G1-G1′ and G2-G2′ are local cross sections from which only the portion of the landslide mass upslope and underneath the proposed access road is analyzed;

- SSA was performed using the Corrected Janbu FOS procedure along Cross Section G-G′ and Simplified Bishop FOS procedure along Cross Sections G1-G1′ and G2-G2′;

- Independent SSA performed by CSA indicate the required lateral loads calculated by CalWest using the Simplified Bishop FOS procedure along Cross Sections G1-G1′ and G2-G2′ result in safety factors which do not meet code compliance when evaluated with a FOS procedure which satisfies both moment and force equilibriums simultaneously. CSA estimates the required lateral loads to be roughly 41% and 38% higher for Cross Sections G1-G1′ and G2-G2′, respectively, when using the more rigorous procedure. See above for recommendations regarding performing SSA using programs which simultaneously satisfy both moment and force equilibriums;

- There was a surcharge load of 250 psf applied at the location of the proposed access road for the analysis of Cross Section G-G′. There was no surcharge load applied for the analyses of Cross Sections G1-G1′ and G2-G2′. The Consultant should re-analyze this cross section using a surcharge load;

- It appears that the reported design loads along Cross Sections G1-G1′ and G2-G2′ were based on an EFP of 30pcf. The Consultant should clarify how an EFP of 30 pcf was derived; and

- The strength parameters of the slide surface are inconsistent between Cross Section G-G′ and Cross Sections G1-G1′ and G2-G2′. The Consultant should provide justification for inconsistencies is shear strength parameters for the same landslide.

Based on the comments and observations provided above, for the engineering geologic model analyzed, **it appears that the design loads calculated for static and pseudo-static stabilization within Region D are inadequate.** We recommend that CalWest consider re-analyzing cross sections G1 and G2 using a more rigorous FOS procedure. If cross sections are reconstructed based on the engineering geologic discussions provided above, then the updated and modified cross sections should be utilized. Material strength parameters and surcharge loads should be made consistent for each of the cross sections. If the design load within this region is still based on an EFP of 30 pcf, then its applicability should be justified, with an explanation regarding how it was determined.

It appears that the mitigation structures (piles) don't extend to the margins of the landslides and stop short by about 10 feet, the Consultant should confirm that the mitigation structures are positioned accurately across the landslide to provide the anticipated support.

<u>2.5 Region E</u>

The design load within Region E was determined by CalWest to be 20.0 kips/ft. This load appears to have been determined by taking the arithmetic average of the required loads along cross sections G3-G3′and G4-G4′.

Based on our review of the analyses performed along cross sections G3-G3′and G4-G4′, we have the following comments and observations:

- For purposes of SSA, cross sections G3-G3′and G4-G4′ are local cross sections from which only the portion of the landslide mass upslope and underneath the proposed access road is analyzed;

- SSA was performed using the Simplified Bishop FOS procedure along G3-G3′and G4-G4′;

- Independent SSA performed by CSA indicate that the required lateral loads calculated by CalWest using the Simplified Bishop FOS procedure along sections G3 and G4 result in safety factors which do not meet code compliance when evaluated with a FOS procedure which satisfies both moment and force equilibriums simultaneously. CSA estimates the required lateral loads to be roughly 18% and 3% higher for sections G3 and G4, respectively, when both equilibriums are satisfied. See above for recommendations regarding performing SSA using programs which simultaneously satisfy both moment equilibrium and force equilibrium; and

- It appears that the reported design loads along cross sections G3-G3′ and G4-G4′ were based on an EFP of 30pcf. The Consultant should clarify how an EFP of 30 pcf was derived.

Based on the comments and observations provided above, for the engineering geologic model analyzed, **it appears that the design loads calculated for static and pseudo-static stabilization within Region E are inadequate.** We recommend that CalWest re-analyze Cross Sections G3 and G4 using a rigorous FOS procedure. If cross sections are reconstructed based on the engineering geologic discussions provided above, then the updated and modified cross sections should be utilized. If the design load within this region is still based on an EFP of 30 pcf, then its applicability should be justified, with an explanation regarding how it was determined.

2.6 Region F

The design load within Region F was determined by CalWest to be 7 kips/ft. This load appears to have been determined based on an analysis of a cross section perpendicular to Cross Section E-E′.

Based on our review of the analyses performed along Cross Section E-E′ we have the following comments and observations:

- For purposes of SSA, Cross Section E-E′ is a global cross section from which the entire length of the landslide mass is analyzed;
- SSA was performed using the Janbu Corrected FOS procedure along Cross Section E-E′.
- It appears that the proposed access road crosses section E-E′ three times along the section. For purposes of SSA, CalWest has represented the combined effect of the piles located at each road crossing with a single point load at the inboard edge of the lowest road crossing and iteratively solved for the necessary magnitude of this point load to reach code compliant FOS for static and pseudo-static loading conditions;
- After performing SSA, CalWest determines that design of pile reinforcement in this region is governed by an EFP of 30 pcf;
- It appears that the total load on all the piles along the 600 feet of access road within this region is calculated by looking at a cross section perpendicular to Cross Section E-E′ at a point where the landslide is roughly 53 feet deep at the center. It appears that CalWest assumes that the pile loads vary linearly along the width of the landslide and are based on the EFP of 30 pcf; and
- The resultant force calculated above is then distributed over the 600 feet of roadway for a design load of 7 kips/ft along the road.

Typical in two-dimensional (2D) SSA, uniform geometry is assumed into and out of the plane of analysis. For example, the 2D SSA performed by CalWest along Cross Section E-E′ requires 21.5 kips per foot *into and out of the plane created by Section E-E′* in order to achieve code compliant FOS. The EFP analyses performed by CalWest appear to calculate the total load acting on a plane created by a cross section perpendicular to Cross Section E-E′. Like the SSA, this total force is distributed *into and out of the plane created by section E-E′*. For design purposes, however, the total load is distributed over the entire length of the roadway (600 feet), *along the length of road alignment*. This methodology is not typical, and is based on assumptions that, if inaccurate, will potentially result in over-stressing of the mitigation structures, including: 1) assuming that the depth of landslide varies uniformly from a hypothesized maximum depth to the margins; and 2) assuming that the one analyzed cross section is the most critical for the entire 600 feet of roadway.

For the engineering geologic model analyzed, **it appears that the design loads calculated for static and pseudo-static stabilization within Region F are inadequate.** We recommend that CalWest provide results of typical SSA and localized retaining wall analyses at several locations along the road within Region F to justify the application of 7 kips per foot *along the road* as the design criteria. If cross sections are reconstructed based on the engineering geologic discussions provided above, then the updated and modified cross sections should be utilized. Again, if design is still governed by the use of 30 pcf EFP, then CalWest should provide justification for the application of this value.

It appears that the mitigation structures (piles) don't extend to the margins of the landslides and stop short by about 20 feet, the Consultant should confirm that the mitigation structures are positioned accurately across the landslide to provide the anticipated support.

3) *Assess whether the structural design (including pile diameters, spacing, embedment, steel reinforcement and orientation, force application, conformity to standards of practice, and ability to adequately resist lateral loads) of proposed remediation structures are appropriate for their intended purposes;*

3) Conclusions and Recommendations - We have reviewed the structural calculations and structural drawings for the above referenced project, and have come up with the following questions that the Engineer of Record needs to clarify.

STRUCTURAL CALCULATIONS:

It appears that the Consultant has designed the piles without applying the code FOS, instead using a "load factor" equal to 1.067 or the ratio of 1.6 (code structural FOS) over 1.5 (SSA FOS). This is not a typically accepted design practice since these two factors of safety may not necessarily be redundant (one applies to uncertainties in strengths, distributions and behaviors of structural materials and the other applies to uncertainties in subsurface conditions, soil parameters and limitations inherent in slope stability analyses, etc.). The Consultant should redesign the piles applying the code FOS to the design load determined from the SSA that results in a FOS of 1.5.

With respect to the structural calculations, we recommend that the Consultant address the following items:

1. Page 5: Show calculation how L2 is determined and how the associated L2 maximum moment is determined; this applies to all piles.

2. Page 5: Show calculation how embedment is determined as noted by calculation on Page 115.

3. Page 5: Clarify why piles in the 47-51 foot depth range have smaller embedment, though larger moments, as compared to the piles in the 42-44 foot depth range that have smaller moments and deeper embedment; this applies to all piles.

4. Page 6: Clarify by hand calculation how this spreadsheet works.

5. Page 11: Revise PCACOL interaction diagram to include dead and live load from roadway with moment from landslide; revise reinforcing as necessary; typical all piles.

6. Page 11: It appears that the piles are over-reinforced as flexural members, verify and revise as necessary; typical all piles.

7. Page 26: Clarify why pile embedment calculation does not match structural drawings.

8. Page 26: Clarify why pile summary sheet on Page 4 indicates pile diameter to be 3'-6", while structural drawings indicate 3'0".

9. Page 68: Clarify why pile moment does not match PCACOL and Pile Summary on Page 4.

10. Page 93: Provide load combination 1.2D+1.6+1.6H, and 0.9D+1.0E+1.6H per ACI 318 in PCACOL.

11. Page 93: Show calculation that the shear wall pile meets ACI shear wall requirements §11.10 and §14, and boundary zone requirements §21.7.

12. Page 118: The sketch indicates four concrete beams that run parallel within the roadway deck that supports the roadway deck. It appears that there should be a transfer girder located perpendicular to the roadway at the piles to support these concrete beams; clarify and revise calculations and structural drawings as necessary.

13. Page 118-119: Clarify Case I and Case II and associated Enercalc calculations on pages 120 through 125.

14. Page 120: Clarify where the loads used in the program are from.

15. Page 122-123: Provide calculation for one-way shear, two-way shear, and punching shear for the roadway deck if transfer girder not added.

16. Page 128: Clarify where active pressure, passive pressure, and fire truck loadings are derived from.

17. Page 128: Provide calculation for design of grade beam for shear and torsion.

18. Page 128: Provide calculation how embedment was determined for these retaining walls on a downward slope.

19. Page 128: Clarify if there is any seismic loading for both uphill retaining walls and downward retaining walls.

20. Page 128: Provide calculation for shear transfer of retaining wall to grade beam and grade beam to piles.

21. Page 149: Clarify by hand calculation how spreadsheet works.

22. Page 170-171: Clarify why Enercal key reinforcing is indicated as #4 @ 12.5″ while the structural drawings and calculation page 171 notes #4 @ 16″.

STRUCTURAL DRAWINGS:

1. Sheet S-1: Provide structural drawing index.

2. Sheet S-1: Fill out County of Los Angeles – Structural Observation checklist.

3. Sheet S-2: The enlarged site plan B references the incorrect sheet. Make reference to the appropriate sheet?

4. Sheet S-2: In the Symbol Legend, the detail referenced for the site retaining walls is incorrect. Make reference to the appropriate detail(s).

5. Sheet S-3: On Plan View "A", Detail 3/S9, clarify the spacing of the cast-in-place piles, either in plan or on detail 3/S9.

6. Sheet S-4: On Plan View "C", it indicates that piles 107 and 108 are to be spaced at 9′-0″ while all others are to be spaced at 7′-6″ or 15′-0″. Provide moment, embedment, etc. calculations for 9′-0″ center-to-center pile spacing.

7. Sheet S-6: Detail 1 indicates (12) #9 dowels equally spaced in the pile; provide calculations for shear transfer. It appears the roadway deck is not thick enough to develop the #9 reinforcing dowels from the piles; this applies to all piles, revise drawings as necessary.

8. Sheet S-6: Detail 2, indicates cantilever retaining wall supported of the roadway deck, provide calculations for the connection of the cantilever retaining wall to the cantilever roadway deck.

9. Sheet S-6: Detail 2 indicates the top reinforcing steel as #6 @ 8″ o.c. which is different from Detail 7 which indicates #6 @ 6″ o.c.; clarify and revise.

10. Sheet S-6: Detail 2 Table, indicates "A Slab" reinforcing steel as #5 @ 8″ o.c., which is less than the reinforcing steel indicated in Detail 1 and Detail 7, clarify and revise.

11. Sheet S-6: Detail 2 Note references Detail 1/S5, which is incorrect. Make reference to the appropriate detail(s).

12. Sheet S-6: Detail 2 Table, 2:1 Backfill Angle, "t conc" indicates N/A, the structural calculations references 14″, clarify and revise.

13. Sheet S-6: Detail 8; provide calculations for site retaining wall.

14. Sheet S-7: Provide calculations for shear transfer between roadway deck and the shear wall pile, also refer to question7.

15. Sheet S-8: Clarify how "L2" Zone values on the Pile Schedule were determined and provide calculations.

16. Sheet S-8: For piles 13 through 21 in the Pile Schedule, the shear reinforcement does not match the structural calculations; clarify and revise.

17. Sheet S-8: For piles 36 and 39, the shear demand indicated in the structural calculations on Page 4 is higher than the capacity; clarify and revise.

18. Sheet S-8: For piles 43 through 45 in the Pile Schedule, the shear reinforcing in "L1" Zone does not match the structural calculations summary as noted on Page 4.

19. Sheet S-8: For piles 108 through 183 in the Pile Schedule, the shear reinforcing for "L1" and "L2" Zone indicates #14. The structural calculations indicate #4; revise schedule to indicate the correct reinforcing steel.

20. Sheet S-8: Clarify if there is sufficient spacing between longitudinal reinforcing steel with the proposed concrete mix design; refer to ACI 318 §7.6.3 and §3.3.2.

21. Sheet S-9: Provide calculations for minimum reinforcing steel requirements per ACI 318 for both Detail 1 and Detail 3.

22. Sheet S-9: Detail 1, provide calculations for retaining wall footing/ key, i.e. flexural requirements and minimum steel requirements, etc.

23. Sheet S-9: Detail 1 Table, "Ak" reinforcing steel does not match structural calculations; revise drawings.

24. Sheet S-9: Detail 1 Table, "t conc" for 2:1 backfill angle indicates N/A, the structural calculations indicates 14"; revise drawings.

25. Sheet S-9: Detail 2; provide calculation per ACI Appendix "D" for curtain wall connection to concrete deck and pile; revise detail as required.

26. Sheet S-9: Detail 2; provide calculation for curtain wall and indicated gauge, revise detail as required.

27. Sheet S-9: Details 2 and 3, Provide detail of guardrail and connection, also provide calculations per 2007 CBC and per ACI Appendix "D".

28. Sheet S-9: Detail 3, Provide dimension for retaining wall tiebeam.

29. Sheet S-9: Detail 3, revise detail or show on plan the spacing of the cast-in-place piles.

30. Sheet S-9: Detail 3, Provide calculation (i.e., moment, torsion, etc.) for retaining wall tiebeam for reinforcing steel indicated in table.

31. Sheet S-9: Detail 3, provide calculation for dowels from pile to grade beam, revise drawings as required.

4) *Assess whether the proposed remediation measures will potentially adversely impact slope stability;*

4) Conclusions and Recommendations - As discussed in Section 7.0, grading for the proposed access road, driveways, and building pads will generate approximately 5,000 cubic yards (cy) of net fill materials. Current plans consist of placing fill materials beneath the proposed access road and along the outboard edge of the proposed access road in several places. In addition, three staging areas for the Los Angeles County Fire Department (LACFD) will be constructed. All three staging areas are located within the boundaries of the landslide. The largest of the three staging areas is proposed along the outboard edge of the proposed access road between Station (STA) 46+00 and 50+00. The plans indicate that this staging area will consist of 9,500 cy of fill. The maximum thickness of the proposed staging pad is on the order of 13 feet. Placement of fill materials upon the upslope portion of an existing landslide could potentially have an adverse effect on global slope stability. Therefore, we recommend that CalWest perform appropriate SSA to evaluate the effect of fill placement on the landslide. In particular, the three staging areas should be individually evaluated. The SSA results should be discussed and CalWest should provide recommendations for appropriate mitigation measures if the stability of the slope is adversely impacted.

The Consultant should evaluate the potential for the “non-structural fill” to be susceptible to debris flows during periods of prolonged, and or, intense rainfall. The plans indicate that the non-structural fill will be approximately 16-inches thick; however, there is no indication whether this material will be keyed and benched into the intended slope. The Consultant should clarify how this non-structural fill will be placed, compacted and stabilized to reduce the potential for becoming susceptible to debris flows.

5) *Assess the necessity of fill proposed to be placed between Station 44+60 and Station 52+80 for stability purposes, fire department access and staging, and to evaluate the volume of fill being placed to eliminate off-haul;*

5) **Conclusions and Recommendations** – Based on the electronic files provided to CSA, it is our opinion that the area designated for non-structural fill placement is not adequate for the volume of excess fill expected. Utilizing the grading contours and line work provided by Whitson Engineers on the AutoCad drawing *“Proposed”*, CSA calculated a total surface area of 80,338 square feet designated for placement of non-structural fill containing a volume of 2,600 cubic yards of fill. The fill depicted on the AutoCAD drawing is on average 16 inches thick. According to the plan set “Sweetwater Mesa Road Improvements From STA: 26+70 to 75+53.43”, dated November 4, 2009, the total volume of excess fill expected is 5,250 cubic yards. In addition, the proposed structural piers for the roadway will likely produce at least 6,250 cubic yards of additional spoils (excess fill). Plans for the residences were not provided to CSA; consequently, it is unknown what volume of additional fill may be produced in

conjunction with the foundation elements for the residences and whether that material will be placed/used on each residential site. Consequently, it appears that there could be a significant volume of material not accounted for on-site permanent stockpiling. Furthermore, the volume estimates don't appear to take into account swelling of excavated material and shrinkage of compacted material. Estimates of potential volume changes due to swelling and shrinkage should be provided by the geotechnical engineer based on appropriate laboratory testing and experience with similar materials.

6) *Assess the compatibility and appropriateness of each stabilizing structure/improvement (cuts, fills, retaining walls, drainage, interconnecting piles, and cylindrical piles) necessary for the construction of the 5-lot access road;*

6) **Conclusions and Recommendations** - As discussed in Section 4.0, it appears that Los Angeles County requires three staging areas for the fire department along the proposed access road. It is our understanding that the largest of the three staging areas requires approximately 9,500 cy of fill materials to be placed along the outboard edge of the proposed access road between STA 46+00 and STA 50+00. It appears that the fill for all three staging areas will be placed on existing landslide debris near the upslope portion of the landslide. We did not find evidence of any geotechnical analyses related to this fill placement and are concerned about the possible adverse impact on slope stability.

In our experience, cylindrical piles (piers) can be effective in increasing slope stability if the subsurface has been accurately characterized and the geotechnical analyses have been performed appropriately. Based on our review of the structural design of the cylindrical piles, we understand that the piles have been designed to resist tensile forces primarily in one direction (tension side of steel reinforcement cage). In theory, we agree that such a design could be appropriate and applicable provided the direction of principal lateral earth (landslide) pressures is known within reason. However, in practice, this design requires precision. If the direction of landslide movement has not been adequately determined, or if the contractor installs the steel reinforcement cage at the wrong orientation, the principal tensile forces within the pile could occur in regions of the pile that were not designed to resist tension.

It is also our understanding that Piles 68 – 107 are embedded only 15 feet below the basal shear surface of the landslide and Piles 108 – 185 are embedded only 11 feet below the basal surface of the landslide. There is an element of engineering judgment associated with the design embedment lengths based on the assumed accuracy of the location of the landslide basal shear surface and the potential for landslide movement to transition below the pile reinforcements. In our experience, however, for landslides of this depth and complexity, the embedment depth is usually on the order of 20 feet or greater.

7) *Estimate the extent of additional disturbed areas and volumes of cut and fill necessary if the 1.5:1 slopes must be modified to 2:1;*

7) **Conclusions and Recommendations** - The area of 1.5:1 (horizontal : vertical) cut slopes shown on the plan set "Sweetwater Mesa Road Improvements From STA: 26+70 to 75+53.43", dated November 4, 2009, is located on the inboard side of the proposed roadway from STA 55+60 to STA 61+30, as well as on the north and west side of the Morleigh private driveway and residence. If it is determined that these slopes must be cut to a maximum inclination of 2:1, the additional area disturbed would be 24,000

square feet in the area of the proposed roadway and 2,400 square feet in the area of the Morleigh private driveway and residence. This additional area of cut would produce 5,650 cubic yards of additional spoils in the roadway section and 450 cubic yards of additional spoils in the vicinity of the Morleigh private driveway and residence.

8) Evaluate possible repairs to the pile supported roadway section in the unlikely event of failure due to landslide movement;

8) **Conclusions and Recommendations** – It appears that different landslides or parts of landslides could be moving in different directions, consequently, the reinforcing steel will need to be aligned in the direction of this potential movement. If a pile was oriented in the wrong direction due to installation error and/ or the failure plane differs from what the Geotechnical Engineer has determined, the pile could have insufficient moment capacity due to the special reinforcing steel layout. While not a repair, but more in the line of prevention, it appears that there should be some tolerance (i.e., 15° from centerline each way, for example) in the reinforcing steel layout to provide some redundancy for installation error and/ or the failure plane differing from that determined by the Geotechnical Engineer. In the event of failure due to landslide movement, the existing access road supporting piers would either need to be abandoned or removed and replaced with new piers properly designed to resist additional landslide movement. If failure were caught early enough, then tieback anchors could be installed to support the failing section(s) of roadway. The consultants should recommend a monitoring system and protocol for early warning of potential problems so that they can be addressed early on should they occur.

9) Assess the potential consequences of an unlikely failure of the pile supported roadway section;

9) **Conclusions and Recommendations** - The system currently designed has a factor of safety of 1.5. If a pile was to fail or, more likely, to deform excessively, the forces would then be distributed through the deck to the adjacent piles. As noted in previous structural review comments, punching shear and the method of transferring loads to decking was not provided. The Consultant should describe the mechanism of how these forces would be distributed through the deck, and explain what the remaining safety factor for the pile(s) affected would be.

10) Assess the potential failure mechanisms and repair options of the elevated roadway sections;

10) **Conclusions and Recommendations** - The system currently designed has a factor of safety of 1.5. If a pile was to fail or, more likely, to deform excessively, the forces would then be distributed through the deck to the adjacent piles. As noted in previous structural review comments, punching shear and the method of transferring loads to decking was not provided. The Consultant should describe the mechanism of how these forces would be distributed through the deck, and explain what the remaining safety factor for the pile(s) affected would be.

11) Confirm that roadway grade does not exceed the indicated 18.95 percent, and discuss issues associated with roadways constructed at this inclination;

11) **Conclusions and Recommendations** - The proposed roadway is inclined at 18.95% from STA 31+29.21 to STA 40+39.38, STA 49+15.66 to STA 61+30.26 and STA 67+83.4 to STA 73+04.69. After review of the design plans and electronic drawings, it does not

appear that this inclination is exceeded along the proposed roadway. The profile length of the three sections of roadway listed above are 925 feet, 1,235 feet and 530 feet, respectively. Construction of approximately one half mile of roadway at 18.95% could be difficult and without adequate supervision and inspection could result in a substandard finished product whose design life expectancy would be shortened. In addition to the difficulty in constructing a roadway at such a steep inclination, such a road would put an additional strain on the engines and braking systems of the vehicles that traveled the road frequently. Safety is another issue for steep roadways because the steeper the roadway, the more driving safety issues that could arise.

12) Conduct a thorough spot-checking of calculated quantities for the following using provided topographic information:

a) Volume and area of proposed cuts and fills [1.5:1 (H:V) slopes] of the roadway, residential access roadways, and building pads;

a) **Conclusions and Recommendations** - As discussed above, the areas of 1.5:1 (horizontal : vertical) cut slopes shown on the plan set "Sweetwater Mesa Road Improvements From STA: 26+70 to 75+53.43", dated November 4, 2009, are located on the inboard side of the proposed roadway from STA 55+60 to STA 61+30 as well as on the north and west side of the Morleigh private driveway and residence. The surface area of the proposed 1.5:1 cut inboard of the roadway is 25,500 square feet and the volume of material to be removed is 3,900 cubic yards. The surface area of the proposed 1.5:1 cut located on Morleigh private driveway and residence is 5,200 square feet and the volume of material to be removed is 865 cubic yards.

b) Volume and area of cuts and fills inclined at 2:1 (H:V) for the roadway, residential access roadways, and building pads;

b) **Conclusions and Recommendations** - The total disturbed area for the 2:1 cuts and fill slopes is approximately 357,500 square feet. Within the disturbed area, 40,000 cubic yards of fill will be placed and 32,500 cubic yards of cut excavated. These calculated quantities, combined with the excavation quantities for the 1.5:1 cut slopes, are in conflict with the calculated quantities on the plan set "Sweetwater Mesa Road Improvements From STA: 26+70 to 75+53.43", dated November 4, 2009. The quantities calculated by CSA account for 2,700 cubic yards less cut to be excavated and 4,550 cubic yards more fill to be placed. However, detailed grading plans for the subgrade at the residences were not provided to CSA and could account for minor differences. CSA utilized the finish floor elevations provided for the residential structures to calculate approximate grading quantities in the vicinity of the residences. In addition, since the plans provided to CSA were finished grade contours and not subgrade contours, it is likely that the concrete and structural elements of the roadway sections account for the disparity in fill quantities.

c) Lengths and heights of retaining walls (roadway, residential access roadways, and building pads);

c) **Conclusions and Recommendations** – The retaining wall heights and lengths calculated by CSA were in conformance with those provided by Whitson Engineers. It should be noted that the retaining walls in the area of roadway from STA 51+90 to STA 55+25 on the civil plans dated November 4, 2009, are in conflict with the structural plans dated January 28, 2010. The civil plan's retaining walls are longer than those depicted on the structural plans. The structural plans have additional fill placed on the inboard side of the road eliminating the necessity of a retaining wall. **Also of note is that the design elements on the structural plan appear to be shifted 4.83 feet to the east with respect to the civil plans and underlying topographic base map.**

d) Length of roadway to be stabilized by slab piles and cylinder piles; and

d) **Conclusions and Recommendations** – The sections of roadway to be stabilized within the County of Los Angeles are located from STA 26+76 to STA 31+26 and STA 43+81 to STA 56+86. The profile length of these pier-supported roadway sections are 451.8 feet and 1,319.5 feet, respectively.

e) Length and height of elevated roadway sections;

e) **Conclusions and Recommendations** - It is CSA's understanding that the elevated portions of the roadway are defined by those sections of roadway that refer to Detail 2 on Sheet S-9 of the structural plans dated January 28, 2010. The elevated roadway sections as defined above are located at STA 30+93 to 31+26, STA 51+92 to 52+64 and STA 53+43 to 54+42. The maximum heights for these elevated roadway sections are 4 feet, 18 feet and 10 feet, respectively. The profile lengths of the elevated roadway sections are: STA 30+93 to 31+26, 33 feet; STA51+92 to 52+64, 73.5 feet; and STA 53+43 to 54+42, 100.5 feet. The exact locations of the elevated roadway sections should be more clearly denoted on the design plans; in addition, details should be provided that illustrate how the various retaining walls transition from one to the other.

13) Evaluate the vulnerability of the roadway to geologic hazards;

13) **Conclusions and Recommendations** – The proposed roadway alignment is most vulnerable to potential future reactivation of the existing landslides, seismically induced ground shaking, and rockfalls. As the design now stands, a potential landslide (Landslide 3 in our evaluation) has not been identified. If this landslide is found to be present, then the section of roadway in the vicinity of this feature could be vulnerable to slope instability unless mitigation measures are implemented to properly address this hazard (either avoidance or stabilization). In the event of future prolonged and/or intense rainfall or seismic activity, reactivation of existing landslides could be possible. Because the slope stability analyses did not take into account the possible future presence of groundwater (pore pressures) for any of the landslides, it is difficult to quantify the level of potential risk that the roadway could be exposed to. The consultants should comment on the potential for groundwater to perch and create pore pressures on the relatively impervious basal landslide rupture surface and if it is plausible, how this might affect slope stability. A section of the road from Sta. 27+00 to Sta. 30+00 appears to be susceptible to rockfalls; however, the likelihood of permanent damage to the roadway appears to be low. Mitigation recommendations have been provided to help reduce the risk of rockfalls from impacting the roadway and roadway users; however, to date, roadway design plans have not incorporated these mitigation recommendations.

14) Assess the constructability of the proposed roadway, residential access roadways and building pads;

14) **Conclusions and Recommendations** – Due to the large size of some of the access road piles (up to 8-foot diameter), there are probably only three or four construction companies on the west coast that could construct these structures. However, it is unlikely that any west coast contractors have experience building the Interconnected Pile option. Construction of either the deep large-diameter piles, or the interconnected pile, will likely require slurry to prevent the hole from caving during installation of the cages, multiple cranes to lift and connect cages, and an ample supply of readily available concrete. In order to construct the stabilized sections of roadway, large temporary construction pads will be required. The construction pads will be used for drill rig and crane maneuverability and material storage, and will likely be constructed by side-casting excavation spoils down the slope. The residential access roadways and building pads are within the capabilities and expertise of many local Southern California contractors. The applicant should identify likely locations and sizes of construction staging areas (i.e., temporary construction pads, slurry drying ponds, etc.) and quantities (i.e., slurry and temporary fill pad volumes, etc.) thought to be needed to construct the project.

15) Assess the long term effectiveness and appropriateness of the proposed stabilization elements; and

15) **Conclusions and Recommendations** – While the interconnected pile option in theory could prove effective since it appears to provide more overturning resistance than single piles, we are concerned that this option may not be feasible to construct in this geologic setting (remnant hard rock layers in landslide debris may prove difficult to excavate). Regarding the effectiveness of the cylindrical piles to resist landslide forces, while the concept is a proven concept for stabilizing landslides or portions of landslides, these piles may be insufficiently embedded into the underlying in-place material (in some cases only 11 or 15 feet of embedment). Furthermore, piles resisting 20 or more feet of lateral load (especially landslide loading) are typically braced with tensioned tieback or deadman anchors, although the piles that are very large in diameter may be capable of resisting greater lateral loads. Because many of the piles proposed for the access roadway are not very large in diameter, have shallow embedment depths and are not equipped with anchors, it appears that some of the proposed roadway stabilization piers could prove insufficient for the anticipated landslide or earthquake loadings, have a risk of being over-stressed, and thus may not prove effective in the long term. Satisfactory responses to the structural engineering comments contained in this letter-report would be necessary to provide a final assessment of the appropriateness of the proposed stabilization elements.

16) Identify conceptual level alternative designs and stabilization measures that would reduce grading and wall heights.

16. **Conclusions and Recommendations** – By refining the geologic landslide mapping, it is our preliminary opinion that some reductions in the amount and size of stabilization elements could be realized. It also appears that with some modifications to the roadway alignment, some of the landslide crossings could be either eliminated or reduced, which would reduce the extent of subsurface stabilization elements needed.

17. *Waterline Alignment*

In general, the southern approximately 2,000 feet (from the end of the unimproved roadway, southward to the Ronan residential site) of the waterline alignment extends across relatively steep, west-facing topography that is relatively free of large landslides. In-place bedrock, with some minor, shallow, colluvium-filled swales, was observed for the majority of the alignment. This portion of the alignment is currently undeveloped. The northern approximately 1,500 feet has been partially graded across relatively stable bedrock materials. Some small fillslope failures are located along this alignment, but if the pipeline is located along the inboard edge of the unimproved roadway, these failures should not impact a future pipeline. The northernmost 1,200 feet of the alignment is located within an existing paved private roadway. A large bedrock landslide does appear to be located at the northern end of the alignment; however, two existing residences, utilities, and the roadway are already located atop this landslide. A small landslide is located directly across from the fire hydrant at the north end of the alignment, and has removed a small portion of the edge of the roadway at this location. This small landslide should be addressed prior to installation of the new water line.

SUMMARY

Based on our peer review of the documents and drawings provided, historical aerial photographs, site inspections and analyses, it is our opinion that the applicant's geologic, geotechnical engineering, civil engineering and structural engineering consultants have conducted a great deal of investigative and design work on this challenging project and have developed a reasonable conceptual approach to address these challenges given the site characterization and analyses currently available to them. However, we believe that the information provided to date is insufficient to justify final approval of the project design. The geologic characterization needs to be refined (potentially to the benefit of the scope of the project) and the possibility of an additional large landslide either disproved or taken into consideration in the design. The geotechnical engineering consultant needs to address the refined geologic characterization, perform supplemental laboratory testing to better determine landslide-specific shear strengths and utilize an analysis methodology that satisfies both moment and force equilibriums. The civil and structural engineering consultants will then need to address the refined geologic characterization and geotechnical engineering analysis of that refined characterization utilizing approved design practices. Consequently, we recommend that the applicant's consultants review and satisfactorily address the detailed comments contained in this letter-report prior to the CCC approving the technical aspects of this application for Coastal Development Permits 4-09-056 through 4-09-061 for the Sweetwater Mesa Development Project in Malibu, California.

We trust that this provides the California Coastal Commission with the information that you need at this time. If you have any questions, or need additional information, please contact us.

Very truly yours,

COTTON, SHIRES AND ASSOCIATES, INC.

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POS:JW:DTS:JZ:AM:st

Attachments: References (Documents/Drawings/Electronic Files) Reviewed;
Figure 1 – Aerial Site View;
Figure 2 – CSA Photo-Interpretive Landslides;
Plate 1 – Peer Review Engineering Geologic Map; and
Hohbach-Lewin, Inc., Structural Engineering Peer Review Letter dated February 22, 2010.

REFERENCES (DOCUMENTS/DRAWINGS/ELECTRONIC FILES) REVIEWED

Documents and Drawings:

CalWest Geotechnical Engineering Consultants, May 25, 2007, Geotechnical Engineering Report, Proposed Custom Single-Family Residential Development, APN 4453-005-037 (Lunch), Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, May 25, 2007, Geotechnical Engineering Report, Proposed Custom Single-Family Residential Development, APN 4453-005-018 (Vera), Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, June 1, 2007, Geotechnical Engineering Report, Proposed Custom Single-Family Residential Development, APN 4453-005-092 (Mulryam), Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, June 4, 2007, Geotechnical Engineering Report, Proposed Custom Single-Family Residential Development, APN 4453-005-091 (Morleigh), Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, October 2, 2007, Geotechnical Engineering Report, Proposed Custom Single-Family Residential Development, APN 4453-005-038 (Ronan), Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, December 20, 2007, Geotechnical Engineering Addendum Report, APN 4453-005-018 (Vera), Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, December 27, 2007, Geotechnical Engineering Addendum Report, APN 4453-005-037 (Lunch), Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

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CalWest Geotechnical Engineering Consultants, July 28, 2009, Geotechnical Engineering Letter, Preliminary Grading Plan Review, Proposed Single-Family Residential Development, Malibu Area, County of Los Angeles.

County of Los Angeles, Dept of Public Works, Geotechnical and Materials Engineering Division, October 27, 2008, Soils Engineering Review Sheet, Review of Conceptual Design Pad for Single Family Residence and Access Road.

Kane Geotechnical, October 15, 2007, Sweetwater Mesa Rockfall and Mitigation Study, Los Angeles County.

LC Engineering Group, Inc., October 20, 2009, Structural Analysis and Design: Sweetwater Mesa Rd (Sta 26+70 to 75+52.43), 2930 Sweetwater Mesa Road, Parts 1 and 2.

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Mountain Geology, Inc., December 19, 2007, Addendum Engineering Geologic Report #1 (APN 4453-005-092, Mulryan).

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Mountain Geology, Inc., December 20, 2007, Addendum Engineering Geologic Report #1 (APN 4453-005-091, Morleigh).

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Whitson Engineering, October 21, 2009, Sweetwater Mesa Road Improvement Plans from Sta: 26+70 to 75+53.43.

Whitson Engineering, Revised November 4, 2009, Sweetwater Mesa Road Improvement Plans from Sta: 26+70 to 75+53.43.

Electronic Files (Provided by Whitson Engineering):

Aerial.DWG – Aerial survey in AutoCAD format;

Boundary.DWG – Boundary lines in AutoCAD format;

Proposed.DWG – Linework for the proposed improvements in AutoCAD format;

Lunch-7 Folder – The files within this folder are LDD alignment files for the Lunch Private Access;

Lunch -7-EW Folder – The files within in this folder are LDD surface model files for the Lunch rough grade conditions for the site and residence;

Lunch-7-FG Folder – The files within this folder are LDD surface model files for the Lunch Private Access finish ground condition;

Morleigh-6A-EW Folder - The files within in this folder are LDD surface model files for the Morleigh rough grade conditions for the residence;

Morleigh-6A-Site-EW Folder - The files within in this folder are LDD surface model files for the Morleigh rough grade conditions for the site;

Morleigh-6REV Folder - The files within this folder are LDD alignment files for the Morleigh Private Access;

Morleigh-6REV-FG Folder - The files within this folder are LDD surface model files for the Morleigh Private Access finish ground condition;

Mulryan-5 Folder - The files within this folder are LDD alignment files for the Mulryan Private Access;

Mulryan-5-EW Folder - The files within in this folder are LDD surface model files for the Mulryan rough grade conditions for the site and residence;

Mulryan-5-FG Folder - The files within this folder are LDD surface model files for the Mulryan Private Access finish ground condition;

Ronan-9-EW Folder - The files within this folder are LDD surface model files for the Ronan rough grade conditions for the site and residence;

SWM Aerial Folder – The files within this folder are LDD surface model files for the original ground conditions for the Sweetwater Mesa properties;

SWM Backbone Folder – The files within this folder are LDD surface model files for the Shared Access finish ground condition;

SWM Backbone-7 Folder – The files within this folder are LDD alignment files for the Shared Access;

Vera-(4)a – The files within this folder are LDD alignment files for the Vera Private Access;

Vera-3-FG – The files within this folder are LDD surface model files for the Vera Private Access finish ground condition; and

Vera-4-EW – The files within this folder are LDD surface model files for the Vera rough grade conditions for the site and residence.

Base Photo Source: Google Earth Professional 2008

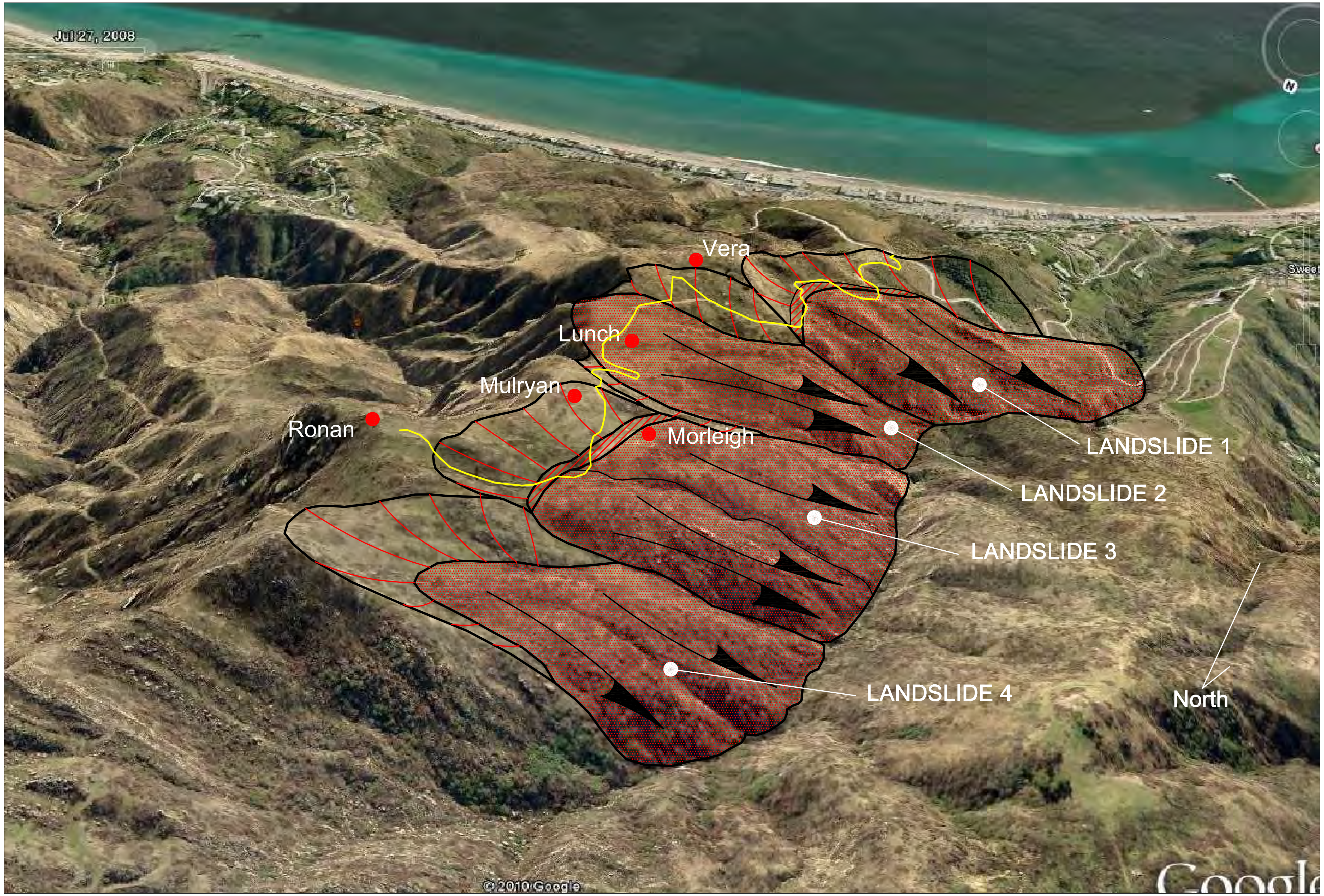
EXPLANATION

- Proposed Residential Site (approx.)
- Proposed Access Road (approx.)

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

AERIAL SITE VIEW
Sweetwater Mesa Development Project
Malibu, California

GEO/ENG BY	SCALE	PROJECT NO.
JW	NA	E5050
APPROVED BY	DATE	FIGURE NO.
JW/POS	March 2010	1



Base Photo Source: Google Earth Professional 2008

EXPLANATION

- Proposed Residential Site (approx.)
- Proposed Access Road (approx.)
- CSA Aerial Photo Landslide Limits
- Possible Landslide Graben Feature

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

CSA PHOTO-INTERPRETIVE LANDSLIDES
Sweetwater Mesa Development Project
Malibu, California

GEO/ENG BY	SCALE	PROJECT NO.
JW	NA	E5050
APPROVED BY	DATE	FIGURE NO.
JW/POS	March 2010	2

Notes:

1. This peer review engineering geologic map is a generalized compilation map based upon Cotton, Shires and Associates, Inc. (CSA) review of historical aerial photographs, limited field mapping, and review of Mountain Geology, Inc. (MGI) geologic and subsurface data. It is not intended to be a project-level, detailed engineering geologic map, but is intended to help clarify our March 8, 2010 peer review report. In preparing this map, we focused primarily on landslide identification along the ridgeline of the proposed development, and did not show mapped faults by MGI or others, or landslides in peripheral areas. For clarity, we have not shown MGI cross section locations or exploratory test pit locations.
2. CSA field reconnaissance geologic mapping was performed in January and February, 2010.
3. CSA performed reconnaissance mapping of the entire proposed waterline alignment; however, only a portion of that alignment is shown on this map.
4. The site topographic base map is from Whitson Engineers, and the proposed residential locations and roadway layouts were approximated from Whitson Engineers drawings.

EXPLANATION

EARTH MATERIALS

- Af — Artificial Fill
- Qsw — Slopewash
- Landslide Debris
  - Als: Active Landslide
  - Qls_s: ShallowLandslide
  - Qls_d: Deep Landslide
- Tcv — Conejo Volcanics
- Tv — Vaqueros Formation

MAP SYMBOLS

- Existing Paved Road
- Existing Dirt Trail
- Proposed Road
- Property Boundary
- Drainage
- Fill Slope
- Cut Slope
- Headscarp
- Landslide Debris
- Landslide Toe
- Escarpment, Sharp Break in Slope
- Graben
- W — W Proposed Water Main Alignment
- Proposed Home Site
- 25° Strike And Dip of Sedimentary Bedrock
- Fault/Shear Orientation
- B-13 Exploratory Borings By Mountain Geology, Inc., 2006 and 2007

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

PEER REVIEW ENGINEERING GEOLOGIC MAP
Sweetwater Mesa Development Project
Malibu, California

GEO/ENG BY	SCALE	PROJECT NO.
JW/POS	1"=200'	E5050
APPROVED BY	DATE	PLATE NO.
JW/POS	March 2010	1

HOHBACH-LEWIN, INC. *STRUCTURAL ENGINEERS*

"Timely Solutions Based On Timeless Principles"

February 22, 2010

Ms. Lesley Ewing
Senior Costal Engineer
CALIFORNIA COASTAL COMMISSION
45 Fremont Street, Suite 2000
San Francisco, CA 94105-2219

Cotton, Shires and Associates, Inc.
Attn: David Schrier
330 Village Lane
Los Gatos, CA 95030-7128

Project: Sweetwater Mesa Development Project – Civil and Geotechnical Engineering and Engineering Geological Peer Review
Malibu, California
Cotton, Shires and Associates Project No.: P5050
Hohbach-Lewin Project No.: 6890C

Dear Ms. Ewing:

Our office has reviewed the structural plans and calculations of the subject project for their conformance with the 2007 California Building Code (CBC), Structural Specialty Code (based on the 2006 IBC and ASCE 7-05). This review is for the Structural Piles, Elevated Roadway and Retaining Walls only. Comments generated from this review are attached.

This review was based on the following items received by Hohbach-Lewin, Inc.:

Drawings titled – Sweetwater Mesa Road Improvements From STA: 26+70 to 75+53.43, Los Angles County, CA.

Structural drawings: drawing nos. S-1, S-2, S-3, S-4, S-5, S-6, S-7, S-8, S-9, dated January 28, 2010; prepared by LC Engineering Group, Inc. and Whitson Engineers.

Structural calculations: Structural Analysis & Design, Sweetwater Mesa Rd. (Sta. 26+70 to 75+53.43) 2930 Sweetwater Mesa Road, Los Angeles County, CA, dated January 27, 2010, prepared by LC Engineering Group, Inc.

Sincerely,
Hohbach-Lewin, Inc.

Bryan G. Cortnik, S.E.
Associate

260 Sheridan Avenue, Suite 150, Palo Alto, CA 94306 • (650) 617-5930 • Fax (650) 617-5932

PALO ALTO SAN FRANCISCO EUGENE

3) *Assess whether the structural design (including pile diameters, spacing, embedment, steel reinforcement and orientation, force application, conformity to standards of practice, and ability to adequately resist lateral loads) of proposed remediation structures are appropriate for their intended purposes;*

We have reviewed the structural calculations and structural drawings for the above referenced project, and have come up with the following questions that the Engineer of Record needs to clarify.

STRUCTURAL CALCULATIONS:

1. Page 5: Show calculation how L2 is determined and how the associated L2 maximum moment is determined; this applies to all piles.

2. Page 5: Show calculation how embedment is determined as noted by calculation on Page 115.

3. Page 5: Clarify why piles in the 47-51 foot depth range have smaller embedment, though larger moments, as compared to the piles in the 42-44 foot depth range that have smaller moments and deeper embedment; this applies to all piles.

4. Page 6: Clarify by hand calculation how this spreadsheet works.

5. Page 11: Revise PCACOL interaction diagram to include dead and live load from roadway with moment from landslide; revise reinforcing as necessary; typical all piles.

6. Page 11: It appears that the piles are over-reinforced as flexural members; verify and revise as necessary; typical all piles.

7. Page 26: Clarify why pile embedment calculation does not match structural drawings.

8. Page 26: Clarify why pile summary sheet on Page 4 indicates pile diameter to be 3'-6", while structural drawings indicate 3'0".

9. Page 68: Clarify why pile moment does not match PCACOL and Pile Summary on Page 4.

10. Page 93: Provide load combination 1.2D+1.6+1.6H, and 0.9D+1.0E+1.6H per ACI 318 in PCACOL.

11. Page 93: Show calculation that the shear wall pile meets ACI shear wall requirements §11.10 and §14, and boundary zone requirements §21.7.

12. Page 118: The sketch indicates four concrete beams that run parallel within the roadway deck that supports the roadway deck. It appears that there should be a transfer girder located perpendicular to the roadway at the piles to support these concrete beams; clarify and revise calculations and structural drawings as necessary.

13. Page 118-119: Clarify Case I and Case II and associated Enercalc calculations on pages 120 through 125.

14. Page 120: Clarify where the loads used in the program are from.

15. Page 122-123: Provide calculation for one-way shear, two-way shear, and punching shear for the roadway deck if transfer girder not added.

16. Page 128: Clarify where active pressure, passive pressure, and fire truck loadings are derived from.

17. Page 128: Provide calculation for design of grade beam for shear and torsion.

18. Page 128: Provide calculation how embedment was determined for these retaining walls on a downward slope.

19. Page 128: Clarify if there is any seismic loading for both uphill retaining walls and downward retaining walls.

20. Page 128: Provide calculation for shear transfer of retaining wall to grade beam and grade beam to piles.

21. Page 149: Clarify by hand calculation how spreadsheet works.

22. Page 170-171: Clarify why Enercal key reinforcing is indicated as #4 @ 12.5" while the structural drawings and calculation page 171 notes #4 @ 16".

STRUCTURAL DRAWINGS:

1. Sheet S-1: Provide structural drawing index.

2. Sheet S-1: Fill out County of Los Angeles – Structural Observation checklist.

3. Sheet S-2: The enlarged site plan B references the incorrect sheet. Make reference to the appropriate sheet?

4. Sheet S-2: In the Symbol Legend, the detail referenced for the site retaining walls is incorrect. Make reference to the appropriate detail(s).

5. Sheet S-3: On Plan View "A", Detail 3/S9, clarify the spacing of the cast-in-place piles, either in plan or on detail 3/S9.

6. Sheet S-4: On Plan View "C", it indicates that piles 107 and 108 are to be spaced at 9'-0" while all others are to be spaced at 7'-6" or 15'-0". Provide moment, embedment, etc. calculations for 9'-0" center-to-center pile spacing.

7. Sheet S-6: Detail 1 indicates (12) #9 dowels equally spaced in the pile; provide calculations for shear transfer. It appears the roadway deck is not thick enough to develop the #9 reinforcing dowels from the piles; this applies to all piles, revise drawings as necessary.

8. Sheet S-6: Detail 2, indicates cantilever retaining wall supported of the roadway deck, provide calculations for the connection of the cantilever retaining wall to the cantilever roadway deck.

9. Sheet S-6: Detail 2 indicates the top reinforcing steel as #6 @ 8" o.c. which is different from Detail 7 which indicates #6 @ 6" o.c.; clarify and revise.

10. Sheet S-6: Detail 2 Table, indicates “A Slab” reinforcing steel as #5 @ 8” o.c., which is less than the reinforcing steel indicated in Detail 1 and Detail 7, clarify and revise.

11. Sheet S-6: Detail 2 Note references Detail 1/S5, which is incorrect. Make reference to the appropriate detail(s).

12. Sheet S-6: Detail 2 Table, 2:1 Backfill Angle, “t conc” indicates N/A, the structural calculations references 14”, clarify and revise.

13. Sheet S-6: Detail 8; provide calculations for site retaining wall.

14. Sheet S-7: Provide calculations for shear transfer between roadway deck and the shear wall pile, also refer to question7.

15. Sheet S-8: Clarify how “L2” Zone values on the Pile Schedule were determined and provide calculations.

16. Sheet S-8: For piles 13 through 21 in the Pile Schedule, the shear reinforcement does not match the structural calculations; clarify and revise.

17. Sheet S-8: For piles 36 and 39, the shear demand indicated in the structural calculations on Page 4 is higher than the capacity; clarify and revise.

18. Sheet S-8: For piles 43 through 45 in the Pile Schedule, the shear reinforcing in “L1” Zone does not match the structural calculations summary as noted on Page 4.

19. Sheet S-8: For piles 108 through 183 in the Pile Schedule, the shear reinforcing for “L1” and “L2” Zone indicates #14. The structural calculations indicate #4; revise schedule to indicate the correct reinforcing steel.

20. Sheet S-8: Clarify if there is sufficient spacing between longitudinal reinforcing steel with the proposed concrete mix design; refer to ACI 318 §7.6.3 and §3.3.2.

21. Sheet S-9: Provide calculations for minimum reinforcing steel requirements per ACI 318 for both Detail 1 and Detail 3.

22. Sheet S-9: Detail 1, provide calculations for retaining wall footing/ key, i.e. flexural requirements and minimum steel requirements, etc.

23. Sheet S-9: Detail 1 Table, “Ak” reinforcing steel does not match structural calculations; revise drawings.

24. Sheet S-9: Detail 1 Table, “t conc” for 2:1 backfill angle indicates N/A, the structural calculations indicates 14”; revise drawings.

25. Sheet S-9: Detail 2; provide calculation per ACI Appendix “D” for curtain wall connection to concrete deck and pile; revise detail as required.

26. Sheet S-9: Detail 2; provide calculation for curtain wall and indicated gauge, revise detail as required.

27. Sheet S-9: Details 2 and 3, Provide detail of guardrail and connection, also provide calculations per 2007 CBC and per ACI Appendix “D”.

28. Sheet S-9: Detail 3, Provide dimension for retaining wall tiebeam.

29. Sheet S-9: Detail 3, revise detail or show on plan the spacing of the cast-in-place piles.

30. Sheet S-9: Detail 3, Provide calculation (i.e., moment, torsion, etc.) for retaining wall tiebeam for reinforcing steel indicated in table.

31. Sheet S-9: Detail 3, provide calculation for dowels from pile to grade beam, revise drawings as required.

POSSIBLE FAILURE AND REPAIR OPTIONS:

8) *Evaluate possible repairs to the pile supported roadway section in the unlikely event of failure due to landslide movement;*

   If a pile was oriented in the wrong direction due to installation error and/ or the failure plane differs from what the Geotechnical Engineer has determined, the pile could have insufficient moment capacity due to the special reinforcing steel layout. It appears that there should be some tolerance (i.e., 15° from centerline each way, for example) in the reinforcing steel layout to provide some redundancy for installation error and/ or the failure plane differing from that determined by the Geotechnical Engineer.

9) *Assess the potential consequences of an unlikely failure of the pile supported roadway section;*

   The system currently designed has a factor of safety of 1.5. If a pile was to fail or, more likely deform excessively, the forces would then be distributed through the deck to the adjacent piles. Please describe the mechanism of how these forces would be distributed through the deck, and what would be the remaining safety factor for the pile(s) affected.

10) *Assess the potential failure mechanisms and repair options of the elevated roadway sections;*

   The system currently designed has a factor of safety of 1.5. If a pile was to fail or, more likely deform excessively, the forces would then be distributed through the deck to the adjacent piles. Please describe the mechanism of how these forces would be distributed through the deck, and what would be the remaining safety factor for the pile(s) affected.

PLEASE SUBMIT AN ITEMIZED RESPONSE TO THESE ITEMS IN WRITING (IN LETTER FORM), WITH REVISED PLANS AND CALCULATIONS, AS REQUIRED. CLEARLY INDICATE ON THE PLANS AND THE CALCULATIONS ALL REVISIONS MADE BY BUBBLING OR OTHER MEANS.

ATTACHMENT 3

March 26, 2010
P5020
By Email (lewing@coastal.ca.gov)

Ms. Lesley Ewing
Senior Coastal Engineer
CALIFORNIA COASTAL COMMISSION
45 Fremont Street, Suite 2000
San Francisco, California 94105-2219

SUBJECT: **Proposal for Supplemental Civil and Geotechnical Engineering and Engineering Geological Review Services**
RE: Sweetwater Mesa Development Project
Malibu, California

Dear Ms. Ewing:

Cotton, Shires and Associates, Inc. (CSA) is pleased to provide the California Coastal Commission (CCC) with this proposal for supplemental civil and geotechnical engineering and engineering geological review services in support of the CCC's continued review and analysis of the application for Coastal Development Permits 4-09-056 through 4-09-061. This work will be performed as a supplement to the review services we provided under our proposal dated January 21, 2010. The conditions surrounding the project are described in that proposal and will not be repeated herein.

In fulfillment of our scope of work under our January 21, 2010 proposal, we prepared a review report entitled: Summary of Findings - Civil and Geotechnical Engineering and Engineering Geologic Peer Review Services, dated March 8, 2010. The applicant's consultants responded to our review comments in a meeting held at the CCC offices in San Francisco on March 17, 2010. During our meeting with the applicant and applicant's consultants, we were informed that the structural design of the access road support system is being changed from a cantilever pier system to a tied-back pier system. We also agreed that additional subsurface exploration in the form of a trenching, test pits and a large-diameter boring would be performed to clarify the geology in several areas of the project with peer review oversight by our office to observe the findings in the field. The applicant's team of consultants will be addressing the items presented in our March 8, 2010 review report and providing us with a response so we can prepare an updated review report for consideration by the CCC. Consequently, we are proposing the following supplemental scope of work and associated costs for your consideration.

SUPPLEMENTAL SCOPE OF WORK

I. Field Review of Supplemental Subsurface Exploration

A. Subsurface Exploration Review – We will be on site during the supplemental exploratory trenching, test pits and large-diameter drilling to conduct peer review observations of the applicant's consultant's subsurface exploration. We request that the consultant clean and log the exposures prior to calling us to the site to observe them.

II. Engineering Geologic, Geotechnical and Civil/Structural Engineering Assessment and Evaluation of Review Responses

A. Assessments and Evaluations – Based on our review of the supplemental subsurface exploration and responses by the applicant's consultants to our March 8, 2010 review comments, we will assess and evaluate these responses from engineering geologic, geotechnical and civil/structural engineering perspectives.

III. Supplemental Consultation, Reporting and Meetings

A. Supplemental Consultation and Reporting – We will consult with CCC staff on an ongoing basis and we will prepare an updated review letter report which will contain our original and supplemental assessments and evaluations of the consultant's original and supplemental subsurface exploration and review responses. It is intended that this report will be a "stand-alone" document that the CCC can use for its discretionary permit considerations. With some minor changes, our initial review report dated March 8, 2010 is now available also as a "stand-alone" document at the link: http://files.me.com/cottonshires/7thb02 with the password "Sweetwater" (case sensitive). In addition, we attended a meeting in San Francisco on March 17, 2010; consequently as set forth in our original proposal, there will be an add-on to our original budget of $1,500. As with our initial scope of work, CCC staff will be included in all telephone/meeting contacts for this proposed supplemental work.

B. Meeting – In addition to the fieldwork, we have budgeted for attending one additional meeting with CCC staff and the applicant's consultants at our Los Gatos office or at the CCC San Francisco office.

C. Coastal Commission Hearing – Under our original proposal dated January 21, 2010, we already proposed and budgeted for preparing a PowerPoint presentation to summarize our review assessments and evaluations, and for attending a CCC hearing to present our findings. Consequently, there will be no supplemental charges for this task unless it is delayed until after June 2010, in which case there will be supplemental budget needed for reviewing the file and re-preparing for this hearing.

SCHEDULE

Upon our receipt of a signed agreement, we will begin our supplemental peer review services as described above. At this time, we anticipate that the supplemental evaluation and updated report preparation will take approximately two weeks to complete following the applicant's consultants supplemental subsurface exploration and submittal to us of their responses addressing our March 8, 2010 review report. Assuming timely authorization, exploration and response, we will endeavor to provide an updated review report within two weeks of our receipt of the applicant's consultants' responses to our report (possibly by the middle or end of April) and we will be available to participate in the CCC hearing as soon as either May or June of this year.

FEE

We propose to invoice you for our supplemental services on a time-and-expenses basis in accordance with the attached Schedule of Charges. It should be understood that an entirely new structural system is being designed to support the access roadway so that significant geotechnical and structural engineering review of this new design will be required. We estimate that our fees and expenses for Tasks I through III outlined above will be:

Task	Estimated Cost Range
I. – Field Review	**$4,600 to $5,100**
Principal Engineering Geologist (16 hours x $210 = $3,360)	
Senior Staff Engineer/Geologist (8 hours x $130 = $1,040)	
Travel Cost, Mileage, Reproduction, Administrative ($200)	
II. – Assessment and Evaluation of Review Responses	**$15,200 to $16,700**
Senior Principal Engineer (16 hours x $250 = $4,000)	
Principal Engineer/Geologist (16 hours x $210 = $3,360)	
Senior Staff Engineer/Geologist (32 hours x $130 = $4,160)	
Supervising Structural Engineer (20 hours x $175 = $3,500)	
Reproduction, Administrative ($180)	
III. – Consulting, Reporting and Meetings	**$14,800 to $17,300**
A. Meeting in San Francisco 3-17-10 - $1,500	
A. Consultation and Reporting - $10,100 to $11,100	
Senior Principal Engineer (8 hours x $250 = $2,000)	
Principal Engineer/Geologist (10 hours x $210 = $2,100)	
Supervising Structural Engineer (10 hours x $175 = $1,750)	
Senior Staff Engineer/Geologist (16 hours x $130 = $2,080)	
Technical Illustrator (16 hours x $85 = $1,360)	
Reproduction Costs and Admin/Accounting ($510)	
B. Meeting (Prep. and Attendance) - $3,200 to $4,700	
Senior Principal Engineer (5 hours x $250 = $1,250)	
Principal Engineer/Geologist (5 hours x $210 = $1,050)	
Supervising Structural Engineer (5 hours x $175 = $875)	
Travel Cost, Mileage – Los Gatos low, San Francisco high – ($25 to $1,525)	

C. Hearing (Prep. and Attendance) – Already included if in May or June

We will invoice the CCC monthly on a time and expenses basis for Tasks I through III outlined above for an amount ranging from $34,600 to $39,100 and will not exceed $39,100 without prior written authorization. Attendance at additional meetings or hearings or a delay of the hearing (beyond the budgeted two meetings and one hearing in May or June 2010) will be billed on a time-and-expense basis in accordance with our attached Schedule of Charges.

AGREEMENT

If you agree with the Scope of Work, Schedule, and Fee outlined above, as well as the attached Schedule of Charges, Limitations, and Terms, please sign one copy of this proposal and return it to our office or incorporate it as an exhibit into a contract. Receipt of the signed proposal or contract will constitute authorization for us to proceed.

We look forward to providing you with the professional services discussed above. If you have any questions, or need additional information, please contact us.

Very truly yours,

COTTON, SHIRES AND ASSOCIATES, INC.

Patrick O. Shires
President and Senior Principal Geotechnical Engineer, GE 770

____________________________ ________________
Approved and Authorized By Date

POS:JW:st
Attachment: CSA Schedule of Charges, Limitations and Terms

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

ATTACHMENT 5

January 21, 2011
E5050
By email (lewing@coastal.ca.gov)

Ms. Lesley Ewing
Senior Coastal Engineer
CALIFORNIA COASTAL COMMISSION
45 Fremont Street, Suite 2000
San Francisco, California 94105-2219

SUBJECT: **January 2011 Summary of Findings – Engineering Geologic, Geotechnical, Civil and Structural Engineering Peer Review Services**
RE: Sweetwater Mesa Development Project
Malibu, California

Dear Ms. Ewing:

Cotton, Shires and Associates, Inc. (CSA) is pleased to provide the California Coastal Commission (CCC) with this January 2011 summary of our findings in regard to the engineering geologic, geotechnical, civil and structural engineering peer review services we provided in support of the CCC's review and analysis of the application for Coastal Development Permits 4-10-040 through 4-10-045 for the Sweetwater Mesa Development Project in Malibu, California. The project consists of developing five residential lots along with an access road that would extend Sweetwater Mesa Road approximately one mile to the north of its present termination. As you are aware, we previously provided you with a summary of findings dated March 8, 2010, based on submittals reviewed prior to that date. Since that time, we have met with and conducted several conference calls with the applicants' consultants and they have responded to the comments and questions raised in our March 8, 2010 review report, meetings and conference calls with additional submittals. This report represents our response to these submittals.

The new access road is proposed partially in the City of Malibu, but mostly in the County of Los Angeles, California. It is our understanding that our task was to review the geologic, engineering geologic, geotechnical, and supplemental reports and engineering plans and calculations pertaining to the portion of project within the County of Los Angeles for adequacy and compliance with the California Coastal Act policies that require the following: 1) new development in areas of high geologic, flood or fire hazard to be designed in such a way as to minimize risks to life and property; 2) new development must be designed to assure stability and structural integrity; and 3) new development shall consider scenic and visual qualities, protect views along the ocean and scenic coastal areas, minimize the alteration of natural landforms, be visually compatible with the character of surrounding areas, and, where feasible, restore and enhance visual quality in visually degraded areas.

The project-specific requirements include stability review of the portion of the main 5-lot access road that is located within the County of Los Angeles, the individual access roads to the five residential lots, the water line extension to the five properties, and each of the five development areas. Our peer review work has culminated in this final written report summarizing our Scope of Work, Findings, Conclusions and Recommendations, Summary of CCC Requested Scope of Work, Structural Calculations Review, Structural Drawings Review, Limitations and Overall Summary. A list of References (Documents/Drawings/Electronic Files) Reviewed is provided at the back of this report.

SCOPE OF WORK

I. Civil and Geotechnical Engineering and Geologic Evaluation

A. Evaluation of Aerial Photographs – Historical and relatively current aerial photographs were obtained and analyzed with respect to slope stability considerations.

B. Review of Available Data - Published maps and site specific documents pertaining to the project and provided to us by the CCC, including reports, letters, memos and calculations, were reviewed by engineering geologists, civil engineers, geotechnical engineers and structural engineers. The applicant's consultants also provided us with the electronic versions of the drawings in AutoCAD-compatible format to assist us with our review. We used AutoCAD Land Desktop and AutoCAD software to check quantities, etc.

C. Site Reconnaissance - Surficial inspections were completed of the site and vicinity by an engineer and engineering geologists and existing site conditions were noted to formulate a preliminary understanding of the proposed project environment. Inspections of site earth materials and slopes were also conducted, including preliminary engineering geologic mapping of site conditions using provided topographic base maps.

D. Inspection of Trenching and Downhole Logging of Large-Diameter Boring – Our engineering geologist conducted an inspection of trenching performed in a suspected landslide graben area and downhole logging and sampling of an additional large-diameter boring drilled in the vicinity of a recognized deep-seated landslide.

II. Engineering Geologic, Geotechnical and Civil/Structural Engineering Assessment and Evaluation of Site Conditions

A. Assessments and Evaluations – Based on our review of the site conditions, aerial photographs, published maps and site specific documents (including electronic files of the drawings) provided to us, we developed assessments and evaluations to address the CCC's questions, concerns and requests regarding the construction of the proposed roadway, residential access roadways and building pads.

III. Consultation, Reporting and Meetings

A. Consultation and Reporting – We consulted with CCC staff on a regular ongoing basis and we prepared this and previous peer review letter-reports containing our assessments and evaluations of the site conditions, reviewed documents, and addressed each of the CCC's questions, concerns and requests. CCC staff was included in all telephone/meeting/email contacts.

B. Conference Calls – We participated in several conference calls with CCC staff, the applicant's representative and the applicant's consultants.

C. Meetings – In addition to the site inspections, we attended two meetings with CCC staff at our Los Gatos office, one meeting with the applicant's consultants during the initial field trip and two meetings with the applicant's consultants at the CCC offices in San Francisco.

D. Coastal Commission Hearing - We will prepare a PowerPoint presentation to summarize our peer review assessments and evaluations, and attend a CCC hearing to present our findings.

FINDINGS, CONCLUSIONS AND RECOMMENDATIONS

Engineering Geologic Evaluation Introduction – To provide a basis upon which to review the geotechnical and engineering aspects of the proposed development, we performed an engineering geologic evaluation of the project. This evaluation included review of historical stereo-pair aerial photographs (1929, 1952, 1993, and 2000) and historical oblique aerial photographs (1993, 2008, and 2009), and performance of limited engineering geologic field mapping, inspection of trenching and down-hole logging and sampling of a large-diameter boring. Our evaluation also included review of the submitted geologic reports, geologic maps, geologic cross sections, and exploratory borehole and trench logs by the Project Engineering Geologist, Mountain Geology, Inc. (MGI). The fundamental role of the engineering geologist is to first recognize the primary geologic hazards with the potential to impact the proposed development, and second to characterize these geologic hazards so that appropriate geotechnical engineering analyses can be performed. We summarize our evaluation of the engineering geologist's recognition and characterization of the site geologic conditions and geologic hazards, as follows:

Geologic Hazard Recognition – MGI has recognized that landsliding, seismic shaking, rockfalls, and bedrock shattering have the potential to adversely impact the proposed development. MGI has stated that landslide debris underlies the majority of the subject property, and has recommended that mitigation measures be implemented to provide the appropriate required factor of safety for the proposed access road and residences. CSA is in agreement that the majority of the site is underlain by landslide debris, which in general, has been shed westward from the prominent north-south trending ridgeline. Three of the proposed residential structures are located atop the prominent ridgeline on bedrock materials of the Vaqueros Formation. Our review of the proposed development reveals that the Lunch residence is the only proposed living space to be constructed atop landslide debris.

CSA's review of aerial photographs revealed the likely presence of three large landslides and possible presence of a fourth large landslide along the western flank of the ridgeline (we referred to these features as landslides 1 through 4 in our March 8, 2010 "Summary of Findings…" letter), with Landslide 1 on the Vera property (and property to the south of Vera), Landslide 2, a large, mostly evacuated landslide on the Mulryan and Lunch properties, possible Landslide 3 on the Morleigh property, and Landslide 4 north of the

Morleigh property (see Figure 2, CSA Photo-Interpretive Landslides). MGI mapped Landslides 1, 2 and 4, but did not map the features on the Morleigh site as a landslide (CSA possible Landslide 3). Trenching of a possible graben area for Landslide 3 revealed that it was not a landslide, confirming the initial mapping by MGI. In addition to landslides, MGI appears to have adequately recognized other geologic hazards with the potential to adversely impact the proposed site development.

Geologic Characterization – In response to our initial review (CSA, 3/8/10), MGI performed additional geologic field mapping, aerial photograph evaluation, additional subsurface exploration, and refinement of geologic cross sections to portray the site geologic conditions. CSA has reviewed the revised geologic maps, cross sections, and borehole data submitted by MGI, and provides the following summary of the site geologic characterization issues:

Geologic Mapping: In the initial review (CSA, 3/8/10), we noted that MGI had not, in general, differentiated the various types of landslides or slope movements at the site (i.e., shallow landslide, deep landslide, slope wash, talus, etc.), nor had they differentiated the different parts of each landslide (i.e., headscarp, toe, lateral margin, internal slide, etc.). Additionally, the movement directions of the various landslides were not well constrained. We opined that, for certain portions of the proposed development (such as the area we identified as Landslide 2), this could result in overly conservative design assumptions and that planned mitigation elements in some areas could be over-designed. We recommended that MGI incorporate landslide geomorphology into their geologic mapping to help refine the shape, depth interpretations, and direction of the landslides at the site. In response to the review comments, MGI performed additional geologic mapping, geomorphic analysis, subsurface exploration, and refinement of their geologic cross-sections. We also recommended that a well-placed boring along the road alignment above B-9 could provide valuable subsurface information to help refine, and hopefully reduce, the mitigation design. MGI responded to this concern by performing additional geologic mapping and excavating two backhoe-dug test pits (TP-20 and TP-21). The additional data resulted in the re-interpretation of a landslide that had been mapped above boring B-9, refinement of the northern margin of Landslide 2, and refinements to two cross-sections (E-E' and Y-Y').

We also noted that similar refinement of the landslide type and distribution may also have reduced the level of mitigation measures needed at Landslide 1, near Cross Section U-U' and V-V', where the obvious headscarp of the large landslide contained surficial slopewash and talus, and the basal shear surface of the large landslide need not have been extended upslope as initially shown on MGI's U-U' and V-V'. MGI addressed this concern by performing additional geomorphic interpretation, aerial photograph review, and excavation of two backhoe-dug test pits (TP-17 and TP-18) in the headscarp area of this landslide. These data resulted in refinement of the landslide configuration in the headscarp region on MGI's cross-sections through this area.

Subsurface Exploration: A total of 36 large-diameter boreholes, 1 small-diameter borehole, and 16 backhoe test pits were initially excavated by MGI to explore the subsurface conditions at the site. The large-diameter boreholes were drilled to a maximum depth of 70 feet. A total of 14 large-diameter boreholes were drilled within landslide debris, with the basal shear surface of the landslide identified by MGI in the boreholes; however, MGI did not identify striations (and their orientations) on any of the basal shear surfaces.

One of the most pertinent concerns expressed in the initial review was that all large-diameter borehole exploration performed within landslides was conducted near the head of these landslides, which is generally atypical for landslide investigations. However, we also acknowledged the steep and rugged terrain of the site, the difficulty and environmental constraints involved with creating access to the mid and lower portions of the slides, coupled with the fact that the proposed improvements and mitigation elements were only located in the upper reaches of the landslides.

We also offered critique that the descriptions of the landslide basal shear surfaces were short, and were not unique from many other borehole material descriptions. The logs of the exploratory boreholes also did not indicate that hand sampling was performed on any of the basal shear surfaces. Our review of the exploration program revealed an ambitious schedule that resulted in the mobilization, set-up, drilling, sampling, cleaning of the borehole, logging, and backfilling in less than one day per borehole. In our experience, sufficiently detailed landslide logging, sampling, and identification of kinematic markers (i.e., slickensides, striations, etc.) for landslides of this depth typically takes more time. In the initial review, we provided several subsurface exploration recommendations, which are summarized as follows:

- We recommended that additional subsurface exploration be performed along the roadway north of B-9 to characterize the subsurface materials along the steep slope. As discussed above, this was addressed by MGI by two backhoe-dug test pits and additional geologic mapping.

- We recommended that exploratory trenching be considered in the gently sloping area (possible graben) near the proposed Morleigh residence to help determine the presence or absence of landsliding. MGI responded by excavating and logging a 245-foot long trench in this area. The trench exposed significantly faulted bedrock (which is to be expected considering the numerous faults mapped through the ridgeline on regional maps), but no evidence of a landslide graben. The geomorphic expression of the area we identified as Landslide 3 appears to be a function of bedrock orientation coupled with differential weathering and erosion of the underlying bedrock.

- We recommended that additional subsurface exploration be performed downslope of the proposed Lunch residential site. Exploratory borings had been performed in the vicinity of the proposed residence, but no exploration was performed downslope in the vicinity of the roadway; hence, there were no data to constrain the location of the slide plane in the immediate vicinity of the roadway where mitigation elements are to be implemented (e.g., MGI Cross Section G-G'). MGI addressed this concern by drilling, sampling, and downhole logging an additional large diameter boring (B-38) and preparing an additional geologic cross-section (AA-AA').

- We recommended that additional borehole exploration be performed with the intent of obtaining hand samples of the slide plane materials for appropriate laboratory testing, and to further constrain the subsurface landslide geometries where only one positive pick on the basal shear surface has been obtained. MGI responded by drilling an additional borehole (B-38).

Geologic Cross Sections: In the initial review (CSA, 3/8/10), we noted that many of the geologic cross sections could be improved with more refined geologic and geomorphic landslide mapping and subsurface exploration. This was an important consideration because along some of the cross sections, this re-interpretation could result in shallower mitigation elements, such as along Cross Sections B-B′, E-E′, F-F′, U-U′, and V-V′. We noted that other geologic cross sections lacked sufficient subsurface data to accurately constrain the subsurface conditions (i.e., G-G′, K-K′).

The first significant review issue we raised was a recommendation that the geologic cross sections developed by MGI be updated to more accurately reflect the landslide geomorphology at the site. We also suggested that the geologic cross sections should be aligned to the most critical portion of the landslides parallel with the estimated movement direction. After obtaining additional data (mapping, geomorphic interpretation, and subsurface exploration), MGI made revisions to Cross Sections B-B′, E-E′, I-I′, M-M′, U-U′, V-V′ and Y-Y′, and prepared an additional cross section (AA-AA′). MGI opined that even after "sufficiently thorough" logging and description of landslide planes was performed downhole, kinematic/directional data such as striations, polishing, and slickensides were not observed and therefore they concluded these features are not present. We experienced similar difficulties attempting to expose these types of features during the our review logging of MGI's B-38 and concluded that, while they should be present for a landslide that has moved as far as this one, exhuming evidence of them would be very time consuming and require specialized equipment.

A second significant review issue was centered on a lack of subsurface exploration of the landslides in areas other than the headscarp regions (i.e., central and lower portions of the landslides). We initially expressed concern that the interpreted landslide geometries (and underlying bedrock geometry beneath the landslides) on the geologic cross sections downslope of the area explored were poorly constrained, a factor which could significantly effect both back-calculated strengths and forward slope stability analyses. In response to this concern, the project geotechnical engineer, CalWest, indicated that potential variation of landslide geometry downslope of the proposed mitigation elements would not adversely impact the proposed mitigation design. With regard to the potential impact on back-calculated strengths, there was a coordinated effort between the Project Engineering Geologist (MGI) and project geotechnical engineer to model a reasonable range of potential variations in landslide geometry and toe exit points on a critical cross section, in order to address this concern.

<u>Summary of Geologic Characterization</u> – MGI has performed a geologic investigation where valuable surface and subsurface geologic information has been gathered, and specific geologic hazards critical to the performance of the proposed development improvements have been adequately identified. Given the site access constraints, as well as the location of the project improvements within only the upper reaches of landslides where full mitigation is proposed, it is our opinion that MGI has now completed a reasonable job of characterizing the site geologic hazards, limits of landslides, the type of sliding and the depth of the slide planes in the access road corridor where such characterization is most vital.

Geotechnical Engineering Evaluation – Geotechnical engineering aspects of the investigation were conducted by CalWest Geotechnical Consulting Engineers (referred to herein as CalWest). During our initial review, we identified various aspects of the investigation, analysis and design that we believed were not in conformance with typical investigations for a project of this magnitude and complexity. We raised several concerns in our March 8, 2010 report that CalWest responded to with additional testing, analyses and submittals. We expressed a particular concern regarding the basal landslide plane shear strength parameters selected by CalWest from engineering judgment and direct shear strength test results. In our opinion, the method of testing was inappropriate for landslide shear strength evaluation and the direct shear tests were performed on disturbed samples that were not representative of the actual basal shear surface. Furthermore, we felt that the cohesion component selected was too high for an existing landslide. Supplemental torsional ring shear strength testing on a sample of the basal landslide plane later obtained from large-diameter boring B-38 indicated a significantly lower frictional component. Consequently, it was agreed that CalWest would circumvent concerns about the laboratory test results by conducting back-calculation analysis on a range of possible reasonable landslide geometries (since the downslope geometry was poorly constrained by subsurface exploration). A higher cohesion component was deemed acceptable for the overall potential failure plane because a landslide buttressed by the canyon would have to shear through landslide debris across bedding planes and not strictly on a previously sheared surface. For reasonable conservatism, a factor of safety of unity (1.0) was utilized for the back-calculation of shear strength parameters and CalWest determined a friction angle of 15 degrees with cohesion of 200 psf for this scenario. These shear strength parameters were then used for forward analyses and design of access road protection measures.

Geotechnical Subsurface Investigation – It appears that for the entire subsurface investigation program, truly undisturbed samples were not available or used for laboratory testing. It appears that all of the samples used for laboratory testing were either disturbed samples obtained during downhole logging (also called "grab samples"), or were driven by the Kelly bar of the drill rig (although widespread in practice for sampling of large-diameter borings, this is not an ASTM-approved sampling method for obtaining relatively undisturbed samples). However, since the shear strength parameters derived for the basal rupture surface of Landslide 2 are now based on back-calculation as discussed above and not solely on laboratory test results, this is no longer a significant concern with respect to our review.

Laboratory Testing – While we raised several questions regarding the laboratory testing program in our March 8, 2010 report, these concerns were either addressed by additional submittals and laboratory testing or resolved by the reliance on back-calculation for deriving shear strength parameters for the basal rupture surface of Landslide 2.

Slope Stability Analysis and Structural Design – The stabilizing structures were reanalyzed and redesigned in accordance with responses to the recommendations presented in the following sections.

SUMMARY OF CCC REQUESTED SCOPE OF WORK

The following are the 16 itemized requests of the CCC in the order requested (in *italics*), with the corresponding CSA response following the requested scope.

1) *Requested Scope - Evaluate whether the provided material is adequate to determine the aforementioned stability issues and if not, what additional material should be provided.*

1) Conclusions and Recommendations – Based on our initial evaluation of the provided materials, we requested several items be provided in order to allow an adequate basis for evaluating the project stability and other issues. These items were provided and we now concur that the provided material appears to be adequate to evaluate stability issues.

2) *Requested Scope - Assess whether the proposed remediation measures are adequate to provide stability for both static and dynamic loading conditions;*

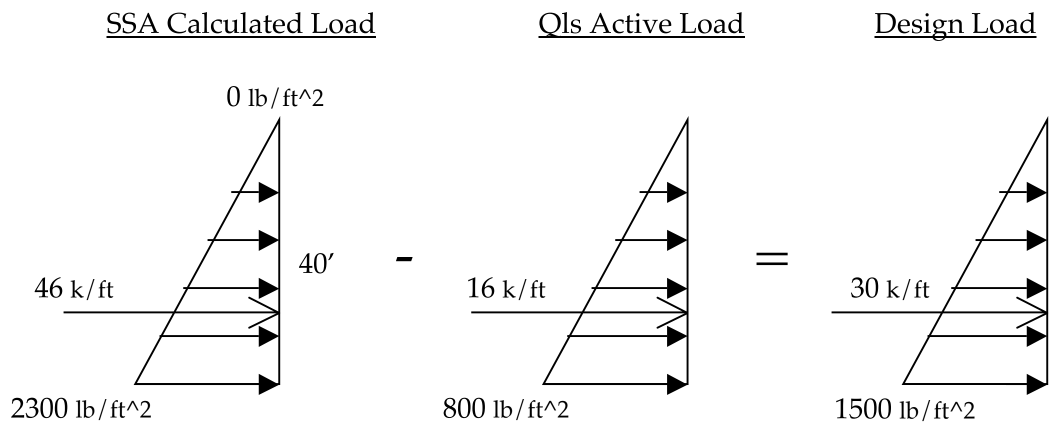
2) Conclusions and Recommendations - Based on our initial evaluation of the provided materials, we requested several items be provided in order to allow an adequate basis for evaluating whether the proposed remediation measures would be sufficient for providing stability for both static and dynamic loading conditions. These items were provided and we now concur that in general, the provided material appears adequate to evaluate both static and dynamic slope stability issues. The following comments summarize our review of CalWest's slope stability analyses:

The methodology/organization of CalWest's Appendix E is as follows: the Appendix was divided into two sections, the first was titled Section 26+70 to 31+40 (Regions A and B from previous report, and Landslide 1 as designated by CSA); and the second was titled 43+70 to 52+80 (Regions D, E and F from previous report and Landslide 2 as designated by CSA). Appendix F contains two figures that illustrate how they determined design loads for the structural design of the shear pins. The Landslide 2 analysis was subsequently updated to utilize the lower shear strengths derived through back-calculation analyses.

Section 26+70 to 31+40 (Landslide 1): Sections U-U', B-B' and V-V' were analyzed with shear pins and the static and pseudostatic (i.e., dynamic loading with horizontal coefficient of 0.15) factors of safety were calculated. For all of the analyses, four methods of analysis were performed: Bishops, Morgenstern-Price, Janbu Corrected and Spencer. For each section, searches were performed for the critical surface using four scenarios: Circular Static, Circular Pseudostatic, Block Static and Block Pseudostatic. Since the access road in the area of Landslide 1 traverses the very upper portion of the landslide where graben formation and backspill of debris has created a basal shear plane with strength properties greater than residual values, higher strength values were utilized by CalWest for Landslide 1 than for Landslide 2.

Section 43+70 to 52+80 (Landslide 2): Sections K-K', AA-AA', G-G', AB-AB', F-F' were analyzed with shear pins and the static and pseudostatic (with horizontal coefficient of 0.15) factors of safety were calculated. For all of the analyses, four methods of analysis were performed: Bishops, Morgenstern-Price, Janbu Corrected and Spencer. For each section, searches were performed for the critical surface using four scenarios: Circular Static, Circular Pseudostatic, Block Static and Block Pseudostatic. For the pseudostatic slope stability analysis, CalWest confirmed that the designed stabilization structures result in an industry accepted standard pseudostatic factor of safety of 1.1 using the back-calculated shear strengths (friction angle of 15 degrees and cohesion of 200 psf) in the analysis of Cross Section G-G'.

Wall Active Forces: For all the analyses, each wall is represented by equivalent fluid pressure active force acting upward to resist the landslide. It appears that this distributed load was varied until the desired design factor of safety was achieved. After determining the required distributed load for stability, the active load from the landslide debris is subtracted leaving only the required resistance from the shear pins. It appears that an equivalent fluid pressure of 30 pcf was used for the landslide debris and applied over 82% of the height (CalWest assumed 3.26 feet of seismic displacement, resulting in a reduced height of 82% of original height). A diagram with example numbers (from Section K-K') is shown below:



The calculated design load was then used to make the figures shown in Appendix F where it was assumed that the required design load varies linearly between sections. This will require close inspection by the Project Geologist during construction to confirm that the subsurface geometry supports this assumption.

3) *Assess whether the structural design (including pile diameters, spacing, embedment, steel reinforcement and orientation, force application, conformity to standards of practice, and ability to adequately resist lateral loads) of proposed remediation structures are appropriate for their intended purposes;*

3) Conclusions and Recommendations - We initially reviewed the structural calculations and structural drawings for the above referenced project, and raised several concerns that the Engineer of Record needed to clarify.

STRUCTURAL CALCULATIONS:

With respect to the structural calculations, the Consultant has satisfactorily addressed the items raised in our March 8, 2010 report.

STRUCTURAL DRAWINGS:

With respect to the structural drawings, the Consultant should satisfactorily address the following items that were raised in our March 8, 2010 report (without an "N" after them) and the following new items (those with an "N" after them). Once these items are addressed, the structures should be appropriate for their intended purpose.

8. Sheet S-6: Detail 2, indicates cantilever retaining wall supported of the roadway deck, provide calculations for the connection of the cantilever retaining wall to the cantilever roadway deck.

Sheet S-6: Detail 2, Table notes "A SLAB", though "A SLAB" is not indicated on in the detail, please clarify and or revise detail as necessary.

25. Sheet S-9: Detail 2; provide calculation per ACI Appendix "D" for curtain wall connection to concrete deck and pile; revise detail as required.

Sheet S-8: Detail 2; the calculation on page 293 assumes 20 PSF for wind loading. Per ASCE 7 t6.4.1.2 and t6.4.2.2, a wind load of 26 PSF is calculated for worse case condition, please clarify and or revise calculations as necessary.

26 Sheet S-9: Detail 2; provide calculation for curtain wall and indicated gauge, revise detail as required.

Sheet S-8: Detail 2; maximum height indicted in detail is 22 feet, where as the calculations indicate a maximum height of 18 feet, revised detail to match calculations or revise calculations to match detail.

27. Sheet S-9: Details 2 and 3, Provide detail of guardrail and connection, also provide calculations per 2007 CBC and per ACI Appendix "D".

Sheet S-8: Details 2 and 3, response indicates calculations to be provided by others at a later date (i.e. deferred submittal). Provide preliminary detail of guardrail connection in accordance with 2007 CBC t1607.7.3 and per ACI Appendix "D", also provide calculation that the roadway deck can resist the guardrail loading.

32N. Sheet S-7: Pile Schedule Cont., Based on the structural calculations P133 should be in the same group as P130 through P132, revised detail accordingly.

33N. Sheet S-7: Clarify how the contractor is to splice the reinforcing cage, revise detail accordingly.

34N. Sheet S-8: Detail 1, Provide calculation for minimum footing steel per ACI t10.5.1 through t10.5.4, for the key. Per the detail "AK" refers to the top steel of the footing and key and "A conc" refers to the bottom steel of the footing, please clarify what the continuous steel in the footing is to be (i.e. reinforcing steel coming out-of-the-plane).

35N. Sheet S-8: Detail 3, Plans indicate a stepped footing; please provide a detail to show how to step the footing of the retaining wall and grade beam.

36N. Sheet S-7: Pile Schedule Cont., Based on the revised structural calculations dated November 16, 2010, specifically calculation Page 5 "Pile Summary Sheet"; revise pile 118 "L2" which reads 11 feet, per the calculations "L2" should read 14 feet, revise schedule as required.

4) *Assess whether the proposed remediation measures will potentially adversely impact slope stability;*

4) Conclusions and Recommendations - According to the design drawings "Sweetwater Mesa Road Improvements From STA: 26+70 to 75+53.43", dated November 16, 2010, grading for the proposed access road, driveways, and building pads will generate approximately 21,600 cubic yards (cy) of net fill materials. Current plans consist of placing surplus fill materials in the non-structural fill area located between STA 44+60 to STA 52+80, this area will accommodate up to 13,950 cy of material. In addition, three staging areas for the Los Angeles County Fire Department (LACFD) will be constructed utilizing 10,000 cy of surplus material. All three staging areas are located within the boundaries of the landslide. The largest of the three staging areas is proposed along the outboard edge of the proposed access road between Station (STA) 46+00 and 50+00. The plans indicate that this staging area will consist of 9,500 cy of fill. The maximum thickness of the proposed staging pad is on the order of 13 feet. Placement of fill materials upon the upslope portion of an existing landslide could potentially have an adverse effect on global slope stability. Therefore, we recommended that CalWest perform appropriate slope stability analysis to evaluate the effect of fill placement on the landslide. CalWest has now analyzed the largest of the three areas and indicates that the stability of the slope below the protective measures will not be significantly adversely impacted (relative negative impact of on the order of 1 to 3 percent).

The Consultant should evaluate the potential for the "non-structural fill" to be susceptible to debris flows during periods of prolonged, and or, intense rainfall. The plans indicate that the non-structural fill will be approximately 4.6 feet thick; however, it is not clear whether this material will be keyed and benched into the intended slope. The Consultant should clarify that the non-structural fill will be keyed and benched, compacted and stabilized to reduce the potential for becoming susceptible to debris flows.

5) *Assess the necessity of fill proposed to be placed between Station 44+60 and Station 52+80 for stability purposes, fire department access and staging, and to evaluate the volume of fill being placed to eliminate off-haul;*

5) **Conclusions and Recommendations** – Based on the design drawings provided to CSA, it is our opinion that the area designated for non-structural fill placement will be adequate for the volume of excess fill expected (13,950 cubic yards), provided the typical section of non-structural fill is approximately 4.6 feet thick. However, if the non-structural fill is only two feet thick, as is stated in the "Supplemental Geotechnical Engineering Letter #6" prepared by CalWest dated November 11, 2010, then the area of non-structural fill will only accommodate approximately 6,050 cubic yards of excess fill. The fill depicted on the Design Drawings prepared by Whitson Engineers titled "Sweetwater Mesa Road Improvements From STA: 26+70 to 75+53.43", dated November 16, 2010 is on average 4.6 feet thick. It is our understanding that this non-structural fill will be placed on the existing scarified ground surface without the benefit of keyways or

benches. It should be understood that placing 4.6 feet of compacted fill on an inclined surface [steeper than 6:1 (H:V)] without the benefit of keyways and benches could result in shallow failures and/or mobilization of the fill. The geotechnical engineer should consider placing the non-structural fill in accordance with the standard fill detail on Sheet C0.3 of the design drawings prepared by Whitson Engineers titled "Sweetwater Mesa Road Improvements From STA: 26+70 to 75+53.43" dated November 16, 2010. If Geotechnical Engineer's intention is to have the non-structural fill placed in accordance with this detail, then this intent should be made clear on the design drawings.

6) *Assess the compatibility and appropriateness of each stabilizing structure/improvement (cuts, fills, retaining walls, drainage, interconnecting piles, and cylindrical piles) necessary for the construction of the 5-lot access road;*

6) **Conclusions and Recommendations** - In our experience, cylindrical piles (piers) can be effective in increasing slope stability if the subsurface has been accurately characterized and the geotechnical analyses have been performed appropriately. Based on our review of the structural design of the cylindrical piles, we understand that the piles have been designed to resist tensile forces primarily in one direction (tension side of steel reinforcement cage). In theory, we agree that such a design could be appropriate and applicable provided the direction of principal lateral earth (landslide) pressures is known within reason. However, in practice, this design requires precision. If the direction of landslide movement has not been adequately determined, or if the contractor installs the steel reinforcement cage at the wrong orientation, the principal tensile forces within the pile could occur in regions of the pile that were not designed to resist tension. Consequently, it will be critical for the Project Geologist to assess landslide characteristics as the piers are constructed to assure that any perceived changes in landslide direction are brought to the attention of the design engineer and adjustments made accordingly to the steel placement. Alternatively, the design could be modified up front to incorporate a range of potential landslide directions, say within a 20- to 30-degree arc as discussed under Item 8 below.

The project plans show that some of the piles are to be embedded only 10 to 17 feet into in-place bedrock. There is an element of engineering judgment associated with the design embedment lengths based on the assumed accuracy of the location of the landslide basal shear surface and the potential for landslide movement to transition below the pile reinforcements. In our experience, however, for landslides of great depth and complexity, the embedment depth is usually on the order of 20 feet or greater.

7) *Estimate the extent of additional disturbed areas and volumes of cut and fill necessary if the 1.5:1 slopes must be modified to 2:1;*

7) **Conclusions and Recommendations** - The area of 1.5:1 (H:V) cut slopes shown on the plan set "Sweetwater Mesa Road Improvements From STA: 26+70 to 75+53.43", dated November 16, 2010, is located on the inboard side of the proposed roadway from STA 55+60 to STA 63+30, as well as on the northern and western side of the Morleigh private driveway and residence. If it is determined that these slopes must be cut to a maximum inclination of 2:1, the additional area disturbed would be approximately 24,000 square feet in the area of the proposed roadway and approximately 2,400 square feet in the area of the Morleigh private driveway and residence. This additional area of cut would produce approximately 5,650 cubic yards of additional spoils in the roadway

section and approximately 450 cubic yards of additional spoils in the vicinity of the Morleigh private driveway and residence.

8) Evaluate possible repairs to the pile supported roadway section in the unlikely event of failure due to landslide movement;

8) **Conclusions and Recommendations** – It appears that different landslides or parts of landslides could be moving in different directions, consequently, the reinforcing steel will need to be aligned in the direction of this potential movement. If a pile was oriented in the wrong direction due to installation error and/ or the failure plane differs from what the Geotechnical Engineer has determined, the pile could have insufficient moment capacity due to the special reinforcing steel layout. While not a repair, but more in the line of prevention, it appears that there should be some tolerance (i.e., 10 to 15° from centerline each way, for example) in the reinforcing steel layout to provide some redundancy for installation error and/ or the failure plane differing from that determined by the Geotechnical Engineer. In the event of failure due to landslide movement, the existing access road supporting piers would either need to be abandoned or removed and replaced with new piers properly designed to resist additional landslide movement. If failure were caught early enough, then tieback anchors could be installed to support the failing section(s) of roadway. The consultants should recommend a monitoring system and protocol for early warning of potential problems so that they can be addressed early on should they occur.

9) Assess the potential consequences of an unlikely failure of the pile supported roadway section;

9) **Conclusions and Recommendations** - The system currently designed has a factor of safety of equal to or greater than 1.5. If a pile were to fail or, more likely, to deform excessively, the forces would then be distributed through the deck to the adjacent piles. As noted in previous structural review comments, the method of transferring loads to decking should be clarified. The Consultant should clarify the mechanism of how these forces would be distributed through the deck, and explain what the remaining safety factor for the pile(s) affected would be.

10) Assess the potential failure mechanisms and repair options of the elevated roadway sections;

10) **Conclusions and Recommendations** - The system currently designed has a factor of safety equal to 1.5 or greater. If a pile were to fail or, more likely, to deform excessively, the forces would then be distributed through the deck to the adjacent piles. As noted in previous structural review comments, the method of transferring loads to decking should be clarified. The Consultant should clarify the mechanism of how these forces would be distributed through the deck, and explain what the remaining safety factor for the pile(s) affected would be.

11) Confirm that roadway grade does not exceed the indicated 18.95 percent, and discuss issues associated with roadways constructed at this inclination;

11) **Conclusions and Recommendations** - The proposed roadway is inclined at 18.95% from STA 31+29.21 to STA 40+39.38, STA 49+15.66 to STA 61+30.26 and STA 67+83.4 to STA 73+04.69. After review of the design plans, it does not appear that this inclination is exceeded along the proposed roadway. The profile lengths of the three sections of roadway listed above are approximately 925 feet, 1,235 feet and 530 feet, respectively.

Construction of approximately one half mile of roadway at 18.95% could be difficult and without adequate supervision and inspection could result in a substandard finished product whose design life expectancy would be shortened. The applicant should confirm that this steep gradient and the distances of this gradient are in compliance with Fire Department and County of Los Angeles requirements for driveways.

12) Conduct a thorough spot-checking of calculated quantities for the following using provided topographic information:

a) Volume and area of proposed cuts and fills [1.5:1 (H:V) slopes] of the roadway, residential access roadways, and building pads;

a) **Conclusions and Recommendations** - As discussed above, the areas of 1.5:1 (H:V) cut slopes shown on the plan set "Sweetwater Mesa Road Improvements From STA: 26+70 to 75+53.43", dated November 16, 2010, are located on the inboard side of the proposed roadway from STA 55+60 to STA 61+30 as well as on the north and west side of the Morleigh private driveway and residence. The surface area of the proposed 1.5:1 cut inboard of the roadway is approximately 25,500 square feet and the volume of material to be removed is approximately 3,900 cubic yards. The surface area of the proposed 1.5:1 cut located on Morleigh private driveway and residence is approximately 5,200 square feet and the volume of material to be removed is approximately 865 cubic yards. These volumes are in accordance with the volume calculations provided on the civil drawings "Sweetwater Mesa Road Improvements From STA: 26+70 to 75+53.43", dated November 16, 2010.

b) Volume and area of cuts and fills inclined at 2:1 (H:V) for the roadway, residential access roadways, and building pads;

b) **Conclusions and Recommendations** - The total disturbed area for the 2:1 cuts and fill slopes is approximately 357,500 square feet. Within the disturbed area, approximately 40,000 cubic yards of fill will be placed and approximately 32,500 cubic yards of cut excavated. These calculated quantities, combined with the non-structural fill area, excavation quantities for the 1.5:1 cut slopes and the excavation quantities for the piers, are in accordance with the approximate calculated quantities on the plan set "Sweetwater Mesa Road Improvements From STA: 26+70 to 75+53.43", dated November 16, 2010.

c) Lengths and heights of retaining walls (roadway, residential access roadways, and building pads);

c) **Conclusions and Recommendations** – The retaining wall heights and lengths calculated by CSA were in conformance with those provided by Whitson Engineers. It should be noted that the retaining walls in the area of roadway from STA 51+90 to STA 55+25 on the civil plans dated November 16, 2010, are in conflict with the structural plans dated November 16, 2010. The civil plan's retaining walls are longer than those depicted on the structural plans. The structural plans have additional fill placed on the inboard side of the road eliminating the necessity of a retaining wall. **Also of note is that the design elements on the structural plan appear to be shifted approximately 4.83 feet to the east with respect to the civil plans and underlying topographic base map.**

d) Length of roadway to be stabilized by slab piles and cylinder piles; and

d) **Conclusions and Recommendations** – The sections of roadway to be stabilized within the County of Los Angeles are located from STA 26+70 to STA 31+40 and STA 43+70 to STA 52+80. The profile length of these pier-supported roadway sections are approximately 476 feet and 924 feet, respectively.

e) Length and height of elevated roadway sections;

e) **Conclusions and Recommendations** - It is CSA's understanding that the elevated portions of the roadway are defined by those sections of roadway that refer to Detail 2 on Sheet S-8 of the structural plans dated November 16, 2010. The elevated roadway sections as defined above are located at STA 30+93 to 31+40 and STA 51+94 to 52+41. The maximum heights for these elevated roadway sections are 4 feet and 18 feet, respectively. The approximate profile lengths of the elevated roadway sections are: STA 30+93 to 31+40, 47.5 feet; and STA51+94 to 52+41, 47.5 feet. The exact locations of the elevated roadway sections should be more clearly denoted on the design plans; in addition, details should be provided that illustrate how the various retaining walls transition from one to the other.

13) Evaluate the vulnerability of the roadway to geologic hazards;

13) **Conclusions and Recommendations** – The proposed roadway alignment is most vulnerable to potential future reactivation of the existing landslides, seismically induced ground shaking, and rockfalls. In the event of future prolonged and/or intense rainfall or seismic activity, reactivation of existing landslides could be possible; however, the roadway should be protected by the recommended system of reinforced concrete piers and once this system is appropriately constructed, it is our opinion that the likelihood of permanent damage to the roadway from these hazards should be low. A section of the road from Sta. 27+00 to Sta. 30+00 appears to be susceptible to rockfalls; however, the likelihood of permanent damage to the roadway appears to be low. Mitigation recommendations have been provided to help reduce the risk of rockfalls from impacting the roadway and roadway users Calwest has performed a seismic static slope stability analysis that demonstrates that the designed shear pins result in an industry accepted standard pseudostatic factor of safety of 1.1 with a seismic coefficient of 0.15.

14) Assess the constructability of the proposed roadway, residential access roadways and building pads;

14) **Conclusions and Recommendations** – Due to the large size of some of the access road piles (up to 5-foot diameter), there are probably half a dozen construction companies on the west coast that could competently construct these structures. Construction of the deep large-diameter piers will likely require casing or slurry to prevent the holes from caving during installation of the cages, multiple cranes to lift and connect cages, and an ample supply of readily available concrete. In order to construct the stabilized sections of roadway, large temporary construction pads will be required. The construction pads will be used for drill rig and crane maneuverability and material storage, and will likely be constructed by side-casting excavation spoils down the slope. The residential access roadways and building pads are within the capabilities and expertise of many local Southern California

contractors. The applicant's consultants should identify likely locations and sizes of construction staging areas (i.e., temporary construction pads, slurry drying ponds, etc.) and quantities (i.e., slurry and temporary fill pad volumes, etc.) that will be needed to construct the project.

15) Assess the long term effectiveness and appropriateness of the proposed stabilization elements; and

15) **Conclusions and Recommendations** – Regarding the long term effectiveness of the cylindrical piles to resist landslide forces, this concept is a proven concept for stabilizing landslides or portions of landslides as long as the piles are sufficiently embedded into the underlying in-place material and not overloaded.

16) Identify conceptual level alternative designs and stabilization measures that would reduce grading and wall heights.

16. **Conclusions and Recommendations** – By refining the geologic landslide mapping, reductions in the amount and size of stabilization elements have been realized. It appears that because of the steepness of the roadway corridor, the ability to devise alternative designs is limited.

17. *Waterline Alignment*

In general, the southern approximately 2,000 feet (from the end of the unimproved roadway, southward to the Ronan residential site) of the waterline alignment extends across relatively steep, west-facing topography that is relatively free of large landslides. In-place bedrock, with some minor, shallow, colluvium-filled swales, was observed for the majority of the alignment. This portion of the alignment is currently undeveloped. The northern approximately 1,500 feet has been partially graded across relatively stable bedrock materials. Some small fillslope failures are located along this alignment, but if the pipeline is located along the inboard edge of the unimproved roadway, these failures should not impact a future pipeline. The northernmost 1,200 feet of the alignment is located within an existing paved private roadway. A large bedrock landslide does appear to be located at the northern end of the alignment; however, two existing residences, utilities, and the roadway are already located atop this landslide.

SUMMARY

Based on our review of the documents and drawings provided, historical aerial photographs, site inspections, logging and analyses, it is our opinion that the applicant's geologic, geotechnical engineering, civil engineering and structural engineering consultants have conducted a great deal of investigative and design work on this challenging project and have developed a reasonable approach to address these challenges given the site characterization and analyses performed Consequently, it is our opinion that, with the exception of some of our more minor concerns and structural details, the applicant's consultants have satisfactorily addressed the comments of our previous report dated March 8, 2010 and satisfactorily performed their work within the standard of care of their respective disciplines. Through the review process and the improved understanding of the geologic conditions that came from the review questions, the proposed structural mitigation plans have changed from using extremely large diameter caissons and the deep "barbell" caissons to the use of large diameter caissons that is now under review. The first design would have limited the work to only two or three teams in the western states that have the skills and competence to undertake this work, whereas there are likely at least half a dozen contractors in the same area that are competent to construct the current design.

We note that the design methodology for this project will rely heavily on field inspections by the Project Geologist during construction to assure that the design assumptions made with respect to landslide directions and depths are born out by the field conditions or that adjustments be made by the Project Geotechnical and Structural Engineers to the design and construction modified accordingly. We also note that there will likely be additional construction staging areas required that have yet to be identified. We conclude that the information provided by the applicant's consultants and itemized in the attached reference list adequately address the proposed project's technical aspects with respect to geology, geotechnical, civil and structural engineering of this application for Coastal Development Permits 4-09-056 through 4-09-061 for the Sweetwater Mesa Development Project in Malibu, California.

LIMITATIONS

Our services consist of professional opinions and conceptual recommendations made in accordance with generally accepted engineering geology, geotechnical, civil and structural engineering principles and practices. No warranty, expressed or implied, or merchantability or 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 provides the California Coastal Commission with the information that you need at this time. If you have any questions, or need additional information, please contact us.

Very truly yours,

COTTON, SHIRES AND ASSOCIATES, INC.

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Senior Principal Civil and Geotechnical Engineer
PE 26397, GE 770

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POS:JW:DTS:JZ:AM:st

Attachments: References (Documents/Drawings/Electronic Files) Reviewed;
Hohbach-Lewin, Inc., Structural Engineering Peer Review Letter dated December 6, 2010.

REFERENCES (DOCUMENTS/DRAWINGS/ELECTRONIC FILES) REVIEWED

Documents and Drawings:

CalWest Geotechnical Engineering Consultants, May 25, 2007, Geotechnical Engineering Report, Proposed Custom Single-Family Residential Development, APN 4453-005-037 (Lunch), Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, May 25, 2007, Geotechnical Engineering Report, Proposed Custom Single-Family Residential Development, APN 4453-005-018 (Vera), Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, June 1, 2007, Geotechnical Engineering Report, Proposed Custom Single-Family Residential Development, APN 4453-005-092 (Mulryan), Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, June 4, 2007, Geotechnical Engineering Report, Proposed Custom Single-Family Residential Development, APN 4453-005-091 (Morleigh), Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, October 2, 2007, Geotechnical Engineering Report, Proposed Custom Single-Family Residential Development, APN 4453-005-038 (Ronan), Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, December 20, 2007, Geotechnical Engineering Addendum Report, APN 4453-005-018 (Vera), Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, December 27, 2007, Geotechnical Engineering Addendum Report, APN 4453-005-037 (Lunch), Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, December 28, 2007, Geotechnical Engineering Addendum Report, APN 4453-005-091 (Morleigh), Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

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CalWest Geotechnical Engineering Consultants, July 14, 2008, Addendum Geotechnical Engineering Report #2, Response to the County of Los Angeles Department of Public Works, Geotechnical and Material Engineering Division, Soils Engineering Review Sheet Miscellaneous Application No 0706150005.

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CalWest Geotechnical Engineering Consultants, July 23, 2008, Addendum Geotechnical Engineering Report #2, Response to the County of Los Angeles Department of Public Works, Geotechnical and Material Engineering Division, Soils Engineering Review Sheet Miscellaneous Application No 0706150004.

CalWest Geotechnical Engineering Consultants, May 1, 2009, Geotechnical Sections and Geologic Map, APN 4453-005-018.

CalWest Geotechnical Engineering Consultants, May 15, 2009, Geotechnical Engineering Supplemental Report, Proposed Compacted "Non-Structural" Fill Areas (Mulryan).

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CalWest Geotechnical Engineering Consultants, July 28, 2009, Geotechnical Engineering Letter, Preliminary Grading Plan Review, Proposed Single-Family Residential Development, Malibu Area, County of Los Angeles.

CalWest Geotechnical Engineering Consultants, May 3, 2010, Supplemental Geotechnical Engineering Letter #1, Additional Clarification of Design Recommendations and Response to California Coastal Commission Review Prepared by Cotton, Shires and Associates, Inc., Proposed Extension of Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, September 13, 2010, Supplemental Geotechnical Engineering Letter #2, Clarification to E-Mail From David Schrier (dschrier@cottonshires.com) Sent Friday, September 10, 2010 5:54 PM on Behalf of The California Coastal Commission, Proposed Extension of Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, September 30, 2010, Supplemental Geotechnical Engineering Letter #3, Additional Comments, Clarification and Response to Items Discussed at the Meeting Held at The California Coastal Commission on September 15, 2010; Proposed Extension of Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, November 1, 2010, Supplemental Geotechnical Engineering Letter #4, Response to Items Discussed Within the Memorandum Prepared by Cotton, Shires and Associates, Dated October 26, 2010 (included in Appendix A), Proposed Extension of Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, November 8, 2010, Supplemental Geotechnical Engineering Letter #5, Response to Discussion Items at The California Coastal Commission Meeting in San Francisco on November 2, 2010 Regarding Sweetwater Mesa Road Extension, Malibu Area, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, November 11, 2010, Supplemental Geotechnical Engineering Letter #6, Proposed Staging Area, Compacted "Non-Structural" Fill, Sweetwater Mesa Road Extension, Malibu Area, County of Los Angeles, California.

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CalWest Geotechnical Engineering Consultants, December 27, 2010, Supplemental Geotechnical Engineering Letter #8, Additional Comments and Clarifications of Stability Analysis and Geotechnical Design Load Criteria, Sweetwater Mesa Road Extension, Malibu, County of Los Angeles, California.

CalWest Geotechnical Engineering Consultants, January 20, Supplemental Geotechnical Engineering Letter #8 (sic), Additional Comments and Clarifications of Stability Analysis and Geotechnical Design Load Criteria, Sweetwater Mesa Road Extension, Malibu, County of Los Angeles, California.

Cotton, Shires and Associates, Inc., March 8, 2010, Summary of Findings – Civil and Geotechnical Engineering and Engineering Geologic Peer Review Services, Sweetwater Mesa Development Project, Malibu, California.

Cotton, Shires and Associates, Inc., October 26, 2010, Remaining Geologic and Geotechnical Concerns (memorandum), Proposed Sweetwater Mesa Development Project, Malibu, California.

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Hohbach-Lewin, Inc., September 15, 2010, Structural Piles, Elevated Roadway and Retaining Walls only, Sweetwater Mesa Development Project, Civil and Geotechnical Engineering and Engineering Geologic Peer Review, Malibu, California.

Hohbach-Lewin, Inc., September 28, 2010, Review of Structural Plans and Calculations, Sweetwater Mesa Development Project, Civil and Geotechnical Engineering and Engineering Geologic Peer Review, Malibu, California.

Hohbach-Lewin, Inc., December 6, 2010, Review of Structural Plans and Calculations, Sweetwater Mesa Development Project, Civil and Geotechnical Engineering and Engineering Geologic Peer Review, Malibu, California, Malibu, California.

Hohbach-Lewin, Inc., January 10, 2011, (Memo) Supplemental Geotechnical Engineering Letter #8, Additional Comments and Clarifications of Stability Analysis and Geotechnical Design Load Criteria, Sweetwater Mesa Road Extension, Malibu, California.

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LC Engineering Group, Inc., September 27, 2009, Engineering Comments on California Coastal Commission's Draft of Scope of Work for Third Party Review, Sweetwater Mesa Development Project.

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LC Engineering Group, Inc., January 27, 2010, Structural Analysis and Design: Sweetwater Mesa Rd (Sta 26+70 to 75+52.43), 2930 Sweetwater Mesa Road.

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LC Engineering Group, Inc., May 28, 2010, Structural Analysis and Design: Sweetwater Mesa Rd (Sta 26+70 to 75+52.43), 2930 Sweetwater Mesa Road.

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Mountain Geology, Inc., May 11, 2007, Report of Engineering Geologic Study – Proposed Custom Single-Family Residential Development (APN 4453-005-092, Mulryan).

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Mountain Geology, Inc., May 11, 2007, Report of Engineering Geologic Study – Proposed Custom Single-Family Residential Development (APN 4453-005-018, Vera), Electronic Copy.

Mountain Geology, Inc., May 11, 2007, Report of Engineering Geologic Study – Proposed Custom Single-Family Residential Development (APN 4453-005-037, Lunch), Electronic Copy.

Mountain Geology, Inc., August 28, 2007, Report of Engineering Geologic Study – Proposed Custom Single-Family Residential Development (APN 4453-005-038, Ronan).

Mountain Geology, Inc., December 18, 2007, Addendum Engineering Geologic Report #1 (APN 4453-005-037, Lunch).

Mountain Geology, Inc., December 19, 2007, Addendum Engineering Geologic Report #1 (APN 4453-005-092, Mulryan).

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Mountain Geology, Inc., July 7, 2008, Addendum Engineering Geologic Report #2 (APN 4453-005-018, Vera) – Electronic Reference Copy.

Mountain Geology, Inc., July 8, 2008, Addendum Engineering Geologic Report #2 (APN 4453-005-091, Morleigh).

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Mountain Geology, Inc., April 23, 2010, Supplemental Engineering Geologic Report #1 – Engineering Geologic Responses to California Coastal Commission Engineering Geologic, Geotechnical Engineering and Civil Engineering Peer Review, APN 4453-005-037, -018, -038, -092, -091 Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

Mountain Geology, Inc., September 14, 2010, Supplemental Engineering Geologic Report #2 – Engineering Geologic Responses to Email from David Schrier and Pat Shires Received on September 10, 2010, APN 4453-005-037, -018, -038, -092, -091 Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

Mountain Geology, Inc., September 30, 2010, Supplemental Engineering Geologic Report #3 – Additional Responses to California Coastal Commission Engineering Geologic, Geotechnical Engineering and Civil Engineering Peer Review, APN 4453-005-037, -018, -038, -092, -091 Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

Mountain Geology, Inc., October 29, 2010, Supplemental Engineering Geologic Report #4 – Additional Responses to California Coastal Commission Engineering Geologic, Geotechnical Engineering and Civil Engineering Peer Review, APN 4453-005-037, -018, -038, -092, -091 Sweetwater Mesa Road, Malibu Area, County of Los Angeles, California.

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Whitson Engineering, March 11, 2009, Driveway, Grading and Drainage Plans for a Single Family Residence (APN 4453-005-092, Mulryan).

Whitson Engineering, March 25, 2009, Driveway, Grading and Drainage Plans for a Single Family Residence (APN 4453-005-091, Morleigh).

Whitson Engineering, April 3, 2009, Driveway, Grading and Drainage Plans for a Single-Family Residence (CDP Submittal Not for Construction), (APN 4453-005-037, Lunch).

Whitson Engineering, April 28, 2009, Contour Grading Exhibit – 2839 Sweetwater Mesa Road (APN 4453-005-037).

Whitson Engineering, August 5, 2009, Driveway, Grading and Drainage Plans for a Single-Family Residence (CDP Submittal Not for Construction), (APN 4453-005-037, Lunch).

Whitson Engineering, August 5, 2009, 2851 U Sweetwater Mesa Road: Driveway, Grading and Drainage Plans for a Single-Family Residence (APN 4453-005-091, Morleigh).

Whitson Engineering, August 5, 2009, 2857 U Sweetwater Mesa Road: Driveway, Grading and Drainage Plans for a Single-Family Residence (APN 4453-005-092, Mulryan).

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Whitson Engineering, October 20, 2009, Sweetwater Mesa Project Summary Analysis Letter, Attn: Leslie Ewing of California Coastal Commission.

Whitson Engineering, October 21, 2009, Sweetwater Mesa Road Improvement Plans from Sta: 26+70 to 75+53.43.

Whitson Engineering, Revised November 4, 2009, Sweetwater Mesa Road Improvement Plans from Sta: 26+70 to 75+53.43.

Whitson Engineering, May 28, 2010, Sweetwater Mesa Road Improvement Plans from Sta: 26+70 to 75+53.43 (Site Plans)

Whitson Engineering, June 2, 2010, Sweetwater Mesa Road Improvement Plans from Sta: 26+70 to 75+53.43 (LACFD/CDP Submittal; Not for Construction).

Whitson Engineering, Revised November 16, 2010, Plan Set, Sweetwater Mesa Road Improvements From STA: 26+70 to 75+53.43, Malibu, Los Angeles County, California.

<u>Electronic Files (Provided by Whitson Engineering):</u>

<u>Aerial.DWG</u> – Aerial survey in AutoCAD format, dated February 16, 2010;

<u>Boundary.DWG</u> – Boundary lines in AutoCAD format, dated February 16, 2010;

<u>Proposed.DWG</u> – Linework for the proposed improvements in AutoCAD format, dated February 16, 2010;

Lunch-7 Folder – The files within this folder are LDD alignment files for the Lunch Private Access;

Lunch -7-EW Folder – The files within in this folder are LDD surface model files for the Lunch rough grade conditions for the site and residence;

Lunch-7-FG Folder – The files within this folder are LDD surface model files for the Lunch Private Access finish ground condition;

Morleigh-6A-EW Folder - The files within in this folder are LDD surface model files for the Morleigh rough grade conditions for the residence;

Morleigh-6A-Site-EW Folder - The files within in this folder are LDD surface model files for the Morleigh rough grade conditions for the site;

Morleigh-6REV Folder - The files within this folder are LDD alignment files for the Morleigh Private Access;

Morleigh-6REV-FG Folder - The files within this folder are LDD surface model files for the Morleigh Private Access finish ground condition;

Mulryan-5 Folder - The files within this folder are LDD alignment files for the Mulryan Private Access;

Mulryan-5-EW Folder - The files within in this folder are LDD surface model files for the Mulryan rough grade conditions for the site and residence;

Mulryan-5-FG Folder - The files within this folder are LDD surface model files for the Mulryan Private Access finish ground condition;

Ronan-9-EW Folder - The files within this folder are LDD surface model files for the Ronan rough grade conditions for the site and residence;

SWM Aerial Folder – The files within this folder are LDD surface model files for the original ground conditions for the Sweetwater Mesa properties;

SWM Backbone Folder – The files within this folder are LDD surface model files for the Shared Access finish ground condition;

SWM Backbone-7 Folder – The files within this folder are LDD alignment files for the Shared Access;

Vera-(4)a – The files within this folder are LDD alignment files for the Vera Private Access;

Vera-3-FG – The files within this folder are LDD surface model files for the Vera Private Access finish ground condition; and

Vera-4-EW – The files within this folder are LDD surface model files for the Vera rough grade conditions for the site and residence.

HOHBACH-LEWIN, INC. STRUCTURAL ENGINEERS

"Timely Solutions Based On Timeless Principles"

December 6, 2010

Ms. Lesley Ewing
Senior Costal Engineer
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Project: Sweetwater Mesa Development Project – Civil and Geotechnical Engineering and Engineering Geological Peer Review
Malibu, California
Cotton, Shires and Associates Project No.: P5050
Hohbach-Lewin Project No.: 6890C

Dear Ms. Ewing:

Our office has reviewed the revised structural calculations and structural drawings for the Sweetwater Mesa Development Project. The following are additional comments as a result of this revision. (The original comment is in italics.)

This review was based on the following items received by Hohbach-Lewin, Inc.:

Drawings titled – Sweetwater Mesa Road Improvements From STA: 26+70 to 75+53.43, Los Angles County, CA.

Structural drawings: drawing nos. S-T, S-1, S-2, S-3, S-4, S-5, S-6, S-7, S-8 dated November 16, 2010; prepared by LC Engineering Group, Inc. and Whitson Engineers.

Structural calculations: Structural Analysis & Design, Sweetwater Mesa Rd. (Sta. 26+70 to 75+53.43) 2930 Sweetwater Mesa Road, Los Angeles County, CA, dated November 16, 2010, prepared by LC Engineering Group, Inc.

PLAN REVIEW COMMENTS:

STRUCTURAL CALCULATIONS:

23N. Structural calculation review is predicated on the approval of the Geotechnical criteria per Cotton Shires and Associates, Inc.

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PALO ALTO SAN FRANCISCO EUGENE

STRUCTURAL DRAWINGS:

8. *Sheet S-6: Detail 2, indicates cantilever retaining wall supported of the roadway deck, provide calculations for the connection of the cantilever retaining wall to the cantilever roadway deck.*

   Sheet S-6: Detail 2, Table notes "A SLAB", though "A SLAB" is not indicated on in the detail, please clarify and or revise detail as necessary.

25. *Sheet S-9: Detail 2; provide calculation per ACI Appendix "D" for curtain wall connection to concrete deck and pile; revise detail as required.*

   Sheet S-8: Detail 2; the calculation on page 293 assumes 20 PSF for wind loading. Per ASCE 7 §6.4.1.2 and §6.4.2.2, I calculate a wind load of 26 PSF for worse case condition, please clarify and or revise calculations as necessary.

26 *Sheet S-9: Detail 2; provide calculation for curtain wall and indicated gauge, revise detail as required.*

   Sheet S-8: Detail 2; maximum height indicted in detail is 22 feet, where as the calculations indicate a maximum height of 18 feet, revised detail to match calculations or revise calculations to match detail.

27. *Sheet S-9: Details 2 and 3, Provide detail of guardrail and connection, also provide calculations per 2007 CBC and per ACI Appendix "D".*

   Sheet S-8: Details 2 and 3, response indicates calculations to be provided by others at a later date (i.e. deferred submittal). Provide preliminary detail of guardrail connection in accordance with 2007 CBC §1607.7.3 and per ACI Appendix "D", also provide calculation that the roadway deck can resist the guardrail loading.

32N. Sheet S-7: Pile Schedule Cont., Based on the structural calculations P133 should be in the same group as P130 through P132, revised detail accordingly.

33N. Sheet S-7: Clarify how the contractor is to splice the reinforcing cage, revise detail accordingly.

34N. Sheet S-8: Detail 1, Provide calculation for minimum footing steel per ACI §10.5.1 through §10.5.4, for the key. Per the detail "AK" refers to the top steel of the footing and key and "A conc" refers to the bottom steel of the footing, please clarify what the continuous steel in the footing is to be (i.e. reinforcing steel coming out-of-the-plane).

35N. Sheet S-8: Detail 3, Plans indicate a stepped footing; please provide a detail to show how to step the footing of the retaining wall and grade beam.

36N. Sheet S-7: Pile Schedule Cont., Based on the revised structural calculations dated November 16, 2010, specifically calculation page 5 "Pile Summary Sheet"; revise pile 118 "L2" which reads 11 feet, per the calculations "L2" should read 14 feet, revise schedule as required.

PLEASE SUBMIT AN ITEMIZED RESPONSE TO THESE ITEMS IN WRITING (IN LETTER FORM), WITH REVISED PLANS AND CALCULATIONS, AS REQUIRED. CLEARLY INDICATE ON THE PLANS AND THE CALCULATIONS ALL REVISIONS MADE BY BUBBLING OR OTHER MEANS.

Sincerely,
Hohbach-Lewin, Inc.

Bryan G. Cortnik, S.E.
Associate