Applicable LCP Policies

RIPARIAN CORRIDOR POLICIES

LUP Policy 5.1.2(j) (Definition of Sensitive Habitat). *An area is defined as a sensitive habitat if it meets one or more of the following criteria: (j) Riparian corridor.*

LUP Policy 5.1.3 Environmentally Sensitive Habitats. *Designate the areas described in 5.1.2 (d) through (j) as Environmentally Sensitive Habitats per the California Coastal Act and allow only uses dependent on such resources in these habitats within the Coastal Zone unless other uses are: (a) consistent with sensitive habitat protection policies and serve a specific purpose beneficial to the public; (b) it is determined through environmental review that any adverse impacts on the resource will be completely mitigated and that there is no feasible less-damaging alternative; and (c) legally necessary to allow a reasonable economic use of the land, and there is no feasible less-damaging alternative.*

LUP Policy 5.1.6 Development Within Sensitive Habitats. *Sensitive habitats shall be protected against any significant disruption of habitat values; and any proposed development within or adjacent to these areas must maintain or enhance the functional capacity of the habitat. Reduce in scale, redesign, or, if no other alternative exists, deny any project which cannot sufficiently mitigate significant adverse impacts on sensitive habitats unless approval of a project is legally necessary to allow a reasonable use of the land.*

LUP Policy 5.1.7 Site Design and Use Regulations. *Protect sensitive habitats against any significant disruption or degradation of habitat values in accordance with the Sensitive Habitat Protection ordinance. Utilize the following site design and use regulations on parcels containing these resources, excluding existing agricultural operations: (a) Structures shall be placed as far from the habitat as feasible. (b) Delineate development envelopes to specify location of development in minor land divisions and subdivisions. (c) Require easements, deed restrictions, or equivalent measures to protect that portion of a sensitive habitat on a project parcel which is undisturbed by a proposed development activity or to protect sensitive habitats on adjacent parcels. (d) Prohibit domestic animals where they threaten sensitive habitats. (e) Limit removal of native vegetation to the minimum amount necessary for structures, landscaping, driveways, septic systems and gardens; (f) Prohibit landscaping with invasive or exotic species and encourage the use of characteristic native species.*

LUP Policy 5.2.1 (Designation of Riparian Corridors and Wetlands). *Designate and define the following as Riparian Corridors:* ***(a)*** *50' from the top of a distinct channel or physical evidence or high water mark of a perennial stream;* ***(b)*** *30' from the top of a distinct channel or physical evidence of the high water mark of an intermittent stream as designated on the General Plan maps and through field inspection of undesignated intermittent and ephemeral streams;* ***(c)*** *100' of the high water mark of a lake, wetland, estuary, lagoon, or natural body of standing water;* ***(d)*** *The landward limit or a riparian woodland plant community;* ***(e)*** *Woodland arroyos within urban areas.*

LUP Policy 5.2.3 (Activities Within Riparian Corridors and Wetlands). *Development activities, land alteration, and vegetation disturbance within riparian corridors and wetlands and required* *buffers shall be prohibited unless an exception is granted per the Riparian Corridor and Wetlands Protection ordinance. As a condition of riparian exception, require evidence of approval for development from the US Army Corps of Engineers, California Department of Fish and Game, and other federal or state agencies that may have regulatory authority over activities with riparian corridors and wetlands.*

LCP IP Section 16.30.060(d-g) (Riparian Exceptions).*Exceptions and conditioned exceptions to the provisions of this chapter may be authorized in accordance with the following procedures:* ***(d)*** *Findings. Prior to the approval of any exception, the Approving Body shall make the following findings: (1) That there are special circumstances or conditions affecting the property; (2) That the exception is necessary for the proper design and function of some permitted or existing activity on the property; (3) That the granting of the exception will not be detrimental to the public welfare or injurious to other property downstream or in the area in which the project is located; (4) That the granting of the exception, in the Coastal Zone, will not reduce or adversely impact the riparian corridor, and there is no feasible less environmentally damaging alternative; and (5) That the granting of the exception is in accordance with the purpose of this chapter, and with the objectives of the General Plan and elements thereof, and the Local Coastal Program Land Use Plan.* ***(e)*** *Conditions. The granting of an exception may be conditioned by the requirement of certain measures to ensure compliance with the purpose of this chapter. Required measures may include, but are not limited to: (1) Maintenance of a protective strip of vegetation between the activity and a stream, or body of standing water. The strip should have sufficient filter capacity to prevent significant degradation of water quality, and sufficient width to provide value for wildlife habitat, as determined by the Approving Body. (2) Installation and maintenance of water breaks. (3) Surface treatment to prevent erosion or slope instabilities. (4) Installation and maintenance of drainage facilities. (5) Seeding or planting of bare soil. (6) Installation and maintenance of a structure between toe of the fill and the high water mark. (7) Installation and maintenance of sediment catch basins.* ***(f)*** *Concurrent Processing of Related Permits. An application for exception may be processed concurrently with applications for discretionary permits required for the activity in question. No ministerial permit(s) for the activities in question shall be issued until an exception has been authorized. All discretionary permits for the activity in question shall include all conditions included in the exception. Where associated discretionary permits are authorized by the Planning Commission or Board of Supervisors, that body shall be authorized to act in place of the Zoning Administrator in considering an application for an exception if the applications are considered concurrently.* ***(g)*** *Expiration. Unless otherwise specified, exceptions issued pursuant to this chapter shall expire one year from the date of issuance if not exercised. Where an exception has been issued in conjunction with a development permit granted pursuant to Chapter 18.10, the exception shall expire in accordance with the provisions of Chapter 18.10.*

LUP Policy 5.2.4 (Riparian Corridor Buffer Setback). *Require a buffer setback from riparian corridors in addition to the specified distances found in the definition of the riparian corridor. This setback shall be identified in the Riparian Corridor and Wetland Protection ordinance and established based on stream characteristics, vegetation and slope. Allow reductions to the buffer setback only upon approval of a riparian exception. Require a 10 foot separation from the edge of the riparian corridor buffer to any structure.*

LUP Policy 5.1.12 (Habitat Restoration With Development Approval). *Require as a condition of development approval, restoration of any area of the subject property which is an identified* *degraded sensitive habitat, with the magnitude of restoration to be commensurate with the scope of the project. Such conditions may include erosion control measures, removal of non-native or invasive species, planting with characteristic native species, diversion of polluting run-off, water impoundment, and other appropriate means, The object of the habitat restoration activities shall be to enhance the functional capacity and biological productivity of the habitat(s) and whenever feasible, to restore them to a condition which can be sustained by natural occurrences, such as tidal flushing of lagoons.*

LUP Policy 5.2.7 Compatible Uses With Riparian Corridors. *Allow compatible uses in and adjacent to riparian corridors that do not impair or degrade the riparian plant and animal systems, or water supply values, such as non-motorized recreation and pedestrian trails, parks, interpretive facilities and fishing facilities. Allow development in these areas only in conjunction with approval of a riparian exception.*

LCP IP Section 16.30.080 (Riparian Corridor Violations). (a) *It shall be unlawful for any person to do cause, permit, aid, abet, suffer or furnish equipment or labor for any development activity within a riparian corridor as defined in Section 16.30.030 unless either (1) a development permit has been obtained and is in effect which authorizes the development activity as an exception; or (2) the activity is exempt from the requirement for a development permit by the provisions of Section 16.30.050 of this chapter.* ***(b)*** *It shall be unlawful for any person to do, cause, permit, aid, abet, suffer or furnish equipment or labor for any development activity within a buffer zone of an arroyo as defined in Section 16.30.030 and as prescribed by the provisions of subsection 16.30.040(b) unless either (1) a development permit has been obtained and is in effect which authorizes the development activity as an exception; or (2) the activity is exempt from the requirement for a development permit by the provisions of Section 16.30.050 of this chapter.* ***(c)*** *It shall be unlawful for any person to exercise a development permit authorizing development activity as an exception without complying with all of the conditions of such permit.* ***(d)*** *It shall be unlawful for any person to knowingly do, cause, permit, aid, abet or furnish equipment or labor for any work in violation of a stop work notice from and after the date it is posted on the site until the stop work notice is authorized to be removed by the Planning Director.*

IP Section 16.32.100(a) Sensitive Habitat Exception Findings. *In granting an exception, the decision-making body shall make the following findings:* **1.** *That adequate measures will be taken to ensure consistency with the purpose of this chapter to minimize the disturbance of sensitive habitats; and* **2.** *One of the following situations exists: (i)The exception is necessary for restoration of a sensitive habitat; or (ii) It can be demonstrated by biotic assessment, biotic report, or other technical information that the exception is necessary to protect public health, safety, or welfare.*

EROSION POLICIES

LUP Policy 6.3.3 (Abatement of Grading and Drainage Problems). *Require as a condition of development approval, abatement of any grading or drainage condition on the property which gives rise to existing or potential erosion problems.*

LCP IP Section 16.22.040 (Erosion Control General Provisions). *No person shall cause or allow the continued existence of a condition on any site that is causing or is likely to cause accelerated erosion as determined by the Planning Director. Such a condition shall be controlled and/or prevented by the responsible person and the property owner by using* *appropriate measures outlined in subsequent sections of this chapter. Additional measures shall be applied if necessary by the responsible person and the property owner. Specific additional measures may be required by the Planning Director. Property owners will be given a reasonable amount of time, as determined by the Planning Director, to control existing problems depending on the severity of the problem, and the extent of necessary control measures. Where feasible, erosion problems shall be controlled no later than the beginning of the next rainy season (October 15).*

LCP IP Sections 16.22.060(a)(d) (Erosion Control Plan). (a) *Prior to issuance of a building permit, development permit or land division, an erosion control plan indicating proposed methods for the control of runoff, erosion, and sediment movement shall be submitted and approved. Erosion control plans may also be required by the Planning Director for other types of applications where erosion can reasonably be expected to occur. The erosion control plan may be incorporated into other required plans, provided it is identified as such. Erosion control plans shall include, as a minimum, the measures required under Sections 16.22.070, 16.22.080, 16.22.090, and 16.22.100 of this chapter. Additional measures or modification of proposed measures may be required by the Planning Director prior to project approval. No grading or clearing may take place on the site prior to approval of an erosion control plan for that activity. Final certification of project completion may be delayed pending proper installation of measures identified in the approved erosion control plan...* ***(d)*** *For major development proposals, the erosion control plans shall be prepared by a registered professional authorized to do such work under state law. For these major projects, detailed plans of all surface and subsurface drainage devices, runoff calculations, and other calculations demonstrating adequacy of drainage structures shall be included. Inspection by the person preparing the plan and certification of proper installation of control measures may be required by the Planning Director. Major proposals include: (1) Subdivisions of more than four lots. (2) Grading in excess of 2,000 cubic yards. (3) Commercial or Industrial Development Permits for new structures; or Residential Development Permit for more than four units. (4) Other projects of a similar nature determined by the Planning Director to cause major land disturbance...*

LCP IP Section 16.22.060 (Erosion Control Plan). *Runoff from activities subject to a building permit, parcel approval or development permit shall be properly controlled to prevent erosion. The following measures shall be used for runoff control, and shall be adequate to control runoff from a ten-year storm:* ***(a)*** *On soils having high permeability (more than two inches/hour), all runoff in excess of predevelopment levels shall be retained on the site. This may be accomplished through the use of infiltration basins, percolation pits or trenches, or other suitable means. This requirement may be waived where the Planning Director determines that high groundwater, slope stability problems, etc., would inhibit or be aggravated by onsite retention, or where retention will provide no benefits for groundwater recharge or erosion control.* ***(b)*** *On projects where onsite percolation is not feasible, all runoff should be detained or dispersed over nonerodible vegetated surfaces so that the runoff rate does not exceed the predevelopment level. Onsite detention may be required by the Planning Director where excessive runoff would contribute to downstream erosion or flooding. Any policies and regulations for any drainage zones where the project is located will also apply.* ***(c)*** *Any concentrated runoff which cannot be effectively dispersed without causing erosion, shall be carried in non-erodible channels or conduits to the nearest drainage course designated for such purpose by the Planning Director or to on-site percolation devices. Where water will be discharged to natural ground or channels,* *appropriate energy dissipaters shall be installed to prevent erosion at the point of discharge.* ***(d)*** *Runoff from disturbed areas shall be detained or filtered by berms, vegetated filter strips, catch basins, or other means as necessary to prevent the escape of sediment from the disturbed area.* ***(e)*** *No earth or organic material shall be deposited or placed where it may be directly carried into a stream, marsh, slough, lagoon, or body of standing water.*

SLOPE STABILITY POLICIES

LUP Policy 6.2.1 (Geologic Assessments for Development On and Near Slopes). *Require a geologic hazards assessment of all development, including grading permits, that is potentially affected by slope instability, regardless of the slope gradient on which the development takes place. Such assessment shall be prepared by County staff under supervision of the County Geologist, or a certified engineering geologist may conduct this review at the applicant's choice and expense.*

LCP IP Section 16.10.050(a)(b)(c)(e) (Requirements for Geologic Assessment). (a) *All development is required to comply with the provisions of this Chapter, specifically including but not limited to, the placement of manufactured homes in the areas designated as SFHAs in the Flood Insurance Study.* ***(b)*** *Hazard Assessment Required. A geologic hazards assessment shall be required for all development activities in the following designated areas: fault zones, one-hundred year floodplains and floodways, and coastal hazard areas, except: as specified in subsections (c) (d) and (e), where a full geologic report will be prepared according to the County Guidelines for Engineering Geologic Reports, or where the County Geologist finds that there is adequate information on file. A geologic hazards assessment shall also be required for development located in other areas of geologic hazard, as identified by the County Geologist or designee, using available technical resources, from environmental review, or from other field review.* ***(c)*** *Geologic Report Required. A full geologic report shall be required: (1) For all proposed land divisions and critical structures and facilities in the areas defined as Earthquake Fault Zones on the state Alquist-Priolo Earthquake Fault Zoning Act maps; (2) Whenever a significant potential hazard is identified by a geologic hazards assessment; (3) For all new reservoirs to serve major water supplies; (4) Prior to the construction of any critical structure or facility in designated fault zones, and (5) When a property has been identified as "Unsafe to Occupy" due to adverse geologic conditions, no discretionary approval or building permit (except approvals and permits that are necessary solely to mitigate the geologic hazard) shall be issued prior to the review and approval of geologic reports and the completion of mitigation measures, as necessary.* ***(e)*** *Additional Report Requirements. Additional information (including but not limited to full geologic, subsurface geologic, hydrologic, geotechnical or other engineering investigations and reports) shall be required when a hazard or foundation constraint requiring further investigation is identified.*

LUP Policy 6.2.2 (Engineering Geology Report). *Require an engineering geology report by a certified engineering geologist and/or soils engineering report when the hazards assessment identifies potentially unsafe geologic conditions in an area of proposed development.*

LCP IP Section 16.10.060 (Geologic Assessment and Report Preparation and Review). (a) *Timing of Geologic Review. Any required geologic, soil, or other technical report shall be completed, reviewed and accepted pursuant to the provisions of this section before any public hearing is scheduled and before any discretionary or development application is approved or* *issued. The County Geologist may agree to defer the date for completion, review, or acceptance of any technical report where the technical information is 1) unlikely to significantly affect the size or location of the project, and 2) the project is not in the area of the Coastal Zone where decisions are appealable to the Coastal Commission. In no event shall such be deferred until after the approval or issuance of a building permit.* ***(b)*** *Report Preparation. The geologic hazards assessment shall be prepared by County staff. Alternately, the assessment may be conducted by a private Certified Engineering Geologist at the applicant's choice and expense. Such privately prepared assessments shall, however, be subject to review and approval as specified in this section.* ***(c)*** *Report Acceptance. All geologic, geotechnical, engineering, and hydrologic reports or investigations submitted to the County as a part of any development application shall be found to conform to County report guidelines. The Planning Director may require an inspection in the field of all exploratory trenches, test pits, and borings excavated for a technical report.* ***(d)*** *Hazard Assessment and Report Expiration. A geologic hazards assessment and all recommendations and requirements given therein, shall remain valid for three years from the date of completion, unless a shorter period is specified in the report by the preparer. A full geologic report shall be valid and all recommendations therein shall remain in effect for three years from the date of completion of the report. The exception to the three year period of validity is where a change in site conditions, development proposal, technical information or County policy significantly affects the technical data, analysis, conclusions or requirements of the assessment or report; in which case the Planning Director may require a new or revised assessment or report.*

LUP Policy 6.2.3 (Conditions for Development and Grading Permits). *Condition development and grading permits based on the recommendations of the Hazard Assessment and other technical reports.*

LUP Policy 6.2.6 (Location of Structures and Drainage Considerations in Unstable Areas). *Require location and/or clustering of structures away from potentially unstable slopes whenever a feasible building site exists away from the unstable areas. Require drainage plans that direct runoff and drainage away from unstable slopes.*

LCP IP Section 16.10.090 (Project Denial). *A development permit or the location of a proposed development shall be denied if the Planning Director determines that geologic hazards cannot be adequately mitigated or the project would conflict with National Flood Insurance Program regulations. Development proposals shall be approved only if the project density reflects consideration of the degree of hazard on the site, as determined from the technical information as reviewed and approved by the Planning Director.*

LCP IP Section 16.10.105 (Notice of Geologic Hazards in Cases of Dangerous Conditions). ***(a)*** *Whenever a site inspection, geologic hazards assessment or full geologic report identifies the presence of a geologic hazard that causes a site, building, structure, or portions thereof to be rendered unsafe or dangerous, then pursuant to the Uniform Code for the Abatement of Structural and Geologic Hazards as amended by subsection (l) of Section 12.10.070 of this Code, the Planning Director may issue a Notice of Geologic Hazard and Order thereon, and may record a Notice of Geologic Hazard with the County Recorder.* ***(b)*** *The Planning Director may initiate abatement procedures pursuant to the Uniform Code for the Abatement of Structural and Geologic Hazards as amended by Section 12.10.070(l) of the County Code.*

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Soils Report with Site Slope Stability Analyses
2000 McGregor, Drive Aptos

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August 24, 2006
2000 McGregor Drive

Dear Mr. Zar, at your request I conducted a field investigation and office analysis for the site at 2000 McGregor Drive during the spring and summer of 2006. The scope of work you requested was to assess site slope stability and the 1996 Grading Plan. The scope of work did not include any assessment of the existing building.

Site Description

The portion of the property investigated is the developed part of the parcel. Figure 1 is a site vicinity map and Plate 1 is a topographic map of the site showing the existing slopes, parking area, terraced areas and the office building.

McGregor Drive provides access to the site. McGregor is a wide two lane frontage road that parallels Highway 1. Highway 1 is four lanes and about 100 feet from the property. Highway 1 and McGregor intersects Borregas Creek at approximately a 90 degree angle with the creek traveling under both roads in culverts.

The site is located on the slopes above Borregas Creek, on the east side of the Creek. The slopes of the creek bank, to the retaining wall above, range in steepness from 40 to 50° degrees; or horizontal to vertical ratio of 0.8 : 1 to 1.2 : 1. Above the retaining wall are a parking lot on the north, and terraced areas to the south. The office building is located along the eastern edge of the site

The upper portions of the slopes are primarily vegetated with grasses and low plants. The lower slopes have low plants, briars, and at some locations low trees.

Site History and the 1996 Grading Plan

A sewer line was constructed on the slope, in the 1950.s, and was later buried under 12 feet to 15 feet of fill. The site was used as a nursery during the 1960.s through the 1980.s which included office / sales area with a bathroom. During the 1990.s the site was used for mixed commercial purposes and living units. In 1996 the County of Santa Cruz contracted your company J.R. Zar Contracting to undertake a grading project to locate and raise the buried sewer manhole and to restore access, via the manhole to the sewer. The project was completed on February 22, 1997 and signed off by the County of Santa Cruz in June of 1997. In 2004 all living units were removed and the property is being used for mixed commercial.

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The County of Santa Cruz 1996-Plan sheets are comprised of two pages; copies are in Appendix 1. The extent of the proposed grading and the location of a proposed retaining wall are also shown on Plate 1.

When the1996-Plan was prepared the exact location of the sewer access manhole was not known as indicated by the note on the plan 'Manhole To be Found & Raised As Neceassry'. Tbe difference between the 1996-Plan location of the manhole and the actual location can be seen on Plate 1 as 13 feet from the assumed location; 10 feet different parallel to Section 3.

The 1996-Plan therefore appears to be conceptual, showing an intended result, rather than being a carefully detailed construction document.

The location of the 1996-Plan retaining wall is therefore also inferred to be conceptual. It is not possible to say at this time what happened during construction. There may have been many reasons, besides the uncertain location of the manhole, why the scope of work changed during construction. The search for the manhole almost certainly required much more excavation than was originally intended or planned to locate the manhole. Soft soils may have been encountered that needed to be replaced in order to gain access to the missing manhole. Simple expediency in completing the project may have resulted in the changed height and location of the retaining wall. The project was completed in 1996.

Reynolds and Associates conducted a site investigation and made recommendation for site grading in their letter report dated April 17, 1996, at the time the 1996-Plan was prepared by the County of Santa Cruz.

Reynolds and Associates conducted construction inspections of the 1996 grading project and concluded (letter of May 27, 1997) 'It is our opinion that the slope reconstruction has been adequately compacted and completed.' Reynolds did not conduct observation of the construction of the retaining wall or final compaction for pavement. Cone Penetrometer (CPT) Soundings were conducted in the parking lot in May of 2006 and identified a contact between upper compacted soils and looser soils below. The depth of the contact appears to fits with the profile Reynolds recommended for benching and placement of compacted. The CPT logs are in Appendix 2.

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Soils Stratigraphy Based on Field Data

Based on the sub-surface soil investigation conducted in 2006 (described in Appendix 2), the site has a history of having fill materials placed (or dumped) on the creek bank and slopes. Underlying the loose surface soils or colluvium is firm native soil or bedrock.

The soil profiles encountered during field work and the nature of the site indicate that present site was formed by:

1) In geologic time the creek was incised by natural processes into native soils comprised of clays, silty-clays, silty-sands, and sands. Firm bedrock is likely to be near the bottom of the creek, and appears to be comprised of silt and sand and is partially indurated, and firm. Native soil forming the surficial layer of the banks and slopes above the creek are/were probably comprised of soft weathered soil, colluvium, and possibly channel and/or flood plain deposits.

2) During more recent times, un-controlled fills of a substantial thickness appears to have been place or dumped onto the creek bank over the native soils. The fill materials appear to be comprised of soils similar to the native soil. It seems possible, if not likely, that the fill could easily have been derived from nearby areas. Spoils from the construction of McGregor Drive or Highway 1, from sewer construction, or from grading of residential or commercial projects could have ended up at the site. From the fieldwork done, it is not possible to tell where the boundary between soft native soils and fill is as all the soils are fine grained and no distinctive marker beds were observed. Also, it was not possible to determine if the fill or native-weathered-topsoils are layered, or form irregular zones without lateral continuity.

3) In 1996 an engineered fill was constructed creating the upper-most portion of the soil profile comprised of compacted silty-clayey sands and silty-sandy clays. These compacted soils are approximately 3 feet to12 feet thick with the thickest part being closest to the top of the slope and being thinner closer to the existing building and McGregor Drive. The soils compacted in 1996 are denser and stronger than the underlying soils; until the depth of the lower firm native sands (described above) are encountered.

4) One soil boring was constructed at the back of the office building. Soils at this location were considerably different than those found at the front of the building. The soils observed were generally lighter in color and contained considerably more sand below a depth of 6 feet. These soils were saturated and soft. Sandy soils were identified during the soil investigation conducted on the adjacent parcel, by Jacobs / Raas and Associates (March 2, 1988, Geotechnical

Investigation First Alarm Building). Reynolds and Associates also identified sandy soils at the southern end of the site during their 1996 investigation. It appears possible that clayey fill soils identified on the opposite side of the building may end toward the southern end of the property and along the property line. Groundwater present in the sandy soils on the flatter adjacent parcel appears to be held back by the finer grained (generally clayey) fill soils on the creek-bank-slope. The finer grained fill soils on the subject property may be acting like a dam.

Existing Site Slope-Stability-- Based on Visual Observations

From March through August of 2006 field observations were made at the site. For analysis, the site has been divided into five cross-sections lines as shown on Plates 1 to 5. The locations where visual observations were made are identified by the cross-section lines.

1) Sections 1 to 2—
   a) In the parking lot there are indications that down-slope creep (or settlement) occurred sometime during the last 10 years. These indications include:
   b) The protrusion of dead-man piers in the parking area. The dead-man are set-back 12 to 18 feet from the face of the existing retaining wall. The protrusion of the piers indicates that the soil around the dead-men has moved down either due to consolidation of the underlying soils, vertical down-slope displacement, or a combination of both.
   c) There are arctuate cracks in the pavement (parallel to the top of slope) starting at the retaining wall and progressing back to near the office building. Most of the cracking and vertical offset is within the space from the retaining wall to the dead-men, with a smaller amount of pavement cracking and vertical offset from the dead-men to the building. It appears that possibly 3 to 12 inches of vertical movement may have occurred at the retaining wall, but quantification is uncertain as the as-built grades are not known.
   d) Sections of the existing retaining-wall lagging were bowed outward, indicating the wall lagging is approaching its capacity to retain the soil behind the wall.
   e) Down-slope of the site, above the creek bank, and near the head wall for the culvert (under McGregor Drive), a surficial slope failure had occurred. The slip is 2 to 3 feet deep and extends about 1/4 to 1/3 of the slope distance up the hill. Other surficial slips may be present in the slopes under the vegetation
   f) At the base of the existing wood retaining wall along the length of the parking area is a concrete footing extends beyond the face of the wood piers about a foot, and extends behind the wall 3 to 4 feet (based on photographs made during construction). Although there is some separation of the soil below the footing from the footing, the separation does not extend more than 6 to 8 inches back and is at most about 1 inch and typically about

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1/4 inch. The lack of greater separation seems to indicates surficial slope failure at the face of the wall has not occurred.

2) Section 2 to 3—
From the parking lot to the sewer manhole the concrete flat-work slopes down-slope slightly at several locations. The slope may have been built-in, or may be due to settlement, or due to slope-movement. The sewer manhole flatwork appears to be intact, with little or no settlement having occurred around the manhole and attached pipe.

3) Section 3 to 4—
From the sewer manhole to across a garden terrace above the existing retaining wall, one of the wood post supporting the retaining wall has completely lost its embedment and an adjacent post has partially lost its embedment. The retaining wall in this area has failed. Gravel backfill behind the failed wall has move down-slope. The embedment of the two post was only about 4 feet, based on the observed bottom of the failed post.

5) Section 4 to 5— evidence of surface movement was not observed.

6) Overall, Section 1 to 5—
The existing wood retaining wall varies in height from ground surface to about 4 1/2 feet, with the typical height being about 3 to 3 1/2 feet to the top of the concrete footing. The wall appears to have been constructed at one time, as the materials used are uniform in type and dimensions. The materials also appear to be uniformly weathered and deteriorated. Most of the wood piers are close to vertical. However, wood lagging between the piers is bowed at many locations.

Based on the field observations, the field data collected, and the laboratory tests conducted the following conclusions can be inferred:

7) The site currently appears to be stable but may have, in the past experienced, slow down-slope creep and/or vertical consolidation of the soil, along the extent of the retaining wall. This creep, and/or settlement, and/or deflection of the retaining wall may continue in the future or the soils may have already stabilized.

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8) The existing retaining wall:
   a) was not adequately constructed at some locations;
   b) may be contributing to the pavement cracking by deflecting outward;
   c) is likely to need reinforcement or replacement at some locations in the near future; and
   d) is likely to need complete replacement at some time in the not to distant future (5 to 10 years) due to the limited life expectancy of wood embedded in soil.

Slope Stability Analyses

Slope stability analyses were conducted to compare mitigation alternatives.

The analyses discussed are for the overall site. The analyses done show that shallow surface failures of the slopes below the retaining wall are possible. Shallow slope failures could undermine the existing retaining wall and cause local failures of the wall. Also, there is insufficient information about the construction of the existing wall to do stability analyses for the portion of the slopes immediately adjacent the retaining wall.

The defining **assumption** for the analyses done is that the site is presently subject to slope movement. Although there is evidence that slope movement occurred in the past as discussed in the previous section of this report, it is not certain the movement is occurring now. Thus the **assumption** is a starting point that may underestimate the true slope. At the present time, even after the very heavy rainfall in the spring of 2006 which triggered many landslides in the County, the subject site does not appear to show signs of further movement. If long term monitoring of the site to assess slope movement is conducted, and the results found that movement was not happening, then the slope models could be adjusted to show at least 10% more stability.

The modeling analyses done compare the slope stability of the existing site to a mitigated site. The proposed mitigation is the installation of a drainage trench system. The affect of the mitigation is to provide a physical short-cut to what probably happens naturally at the site. What presently appears to occur is that during periods of precipitation groundwater accumulates at the site and on the adjacent parcel and then migrates slowly through the fine-grained soils at the site. In August of 2006 the slopes below the site retaining wall were still mostly green indicating that water is still moving through the site and provideding water to the vegetation. The low vegetation on the slopes on the opposite side of the creek is mostly brown, dried, and dead. The proposed drainage trench is expected reduce the total seasonal increase in

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groundwater in the fill, and reduce the duration of higher water levels, thus reducing settlement, and slope movement.

The comparison tool used to assess slope stability is the Factor of Safety (FS). The Factor of Safety is the ratio of resisting forces to driving forces. A Factor of Safety of one (FS=1) indicates mathematically that the forces tending to move the slope down hill are balanced by the forces tending to keep the slope in place. A Factor of Safety of slightly less than one does not mean that the site slopes will fail catastrophically; rather it is likely to mean that some creep down the hill will occur. Similarly, a Factor of Safety of greater than one does not mean that no creep will occur; rather it is likely to mean that slower creep down the hill will occur.
The true Factor of Safety for this site is indeterminate due to a number of factors including:

a) highly variable subsurface soils;
b) difficulty in assessing long-term cohesive strength of soils; and
c) difficulty in assessing the nature of groundwater migration through the soils.

The starting points, for doing the comparative slope analyses in this report, was to derive site models that had FS=1. The existing site conditions were input into a computer program, and then the parameters such as soil strength, subsurface orientation of soil layers, and groundwater elevation were adjusted until an FS =1 was calculated for each section. Very little changing of the data was needed to get to a FS=1 once a uniform method of adjusting field data to drained soil strength was determined. The assumed difference between field strength measured and drained shear strength used in the analyses was to divide the average field strength determined by Cone Penetrometer (CPT) soundings in half. The reduction by 1/2 was based on the laboratory testing done for the project and the modeling results. At locations where collecting CPT data was not possible, the Standard Penetration Test (SPT) data was correlated to the CPT data and shear strengths based on the corresponding values were used. Although this procedure sounds complicated, the data derived from the CPT soundings is virtually continuous through the soil profile, is substantially more accurate than SPT data, and substantially more reproducible, in the my professional opinion. CPT logs are shown in Appendix 2.

Soils strength were adjusted in a manner which tended to minimized the potential benefit of the proposed drainage trench. Specifically cohesion was increased rather than friction angle, or friction angle was decreased to a minimum realistic value before cohesion was decreased.

Based on the models which have a starting points of FS =1, the following improvements occur if a deep drainage trench is installed. The assumed drainage trench is about 16 feet deep in the parking lot area and 12 feet deep elsewhere. Pages 12 to 23 show the stability analyses.

Section 1-- FS=1.00 goes to FS= 1.32
Section 2-- FS=1.00 goes to FS= 1.34
Section 3-- FS=1.00 goes to FS= 1.24
Section 4-- FS= 0.99 goes to FS= 1.24
Section 5-- FS=1.00 goes to FS= 1.11

The least certainty is for Section 5 where no subsurface investigation was conducted. The work done but Reynolds and Associates (1996) and Jacobs and Associates (1988), tends to indicate that the underlying firm bedrock is closer to the surface at the south end of the site and therefore the stability for Section 5 may be better than what has been calculated in this report.

Factors of safety for earthquake loads are higher than 1.2 if the full short-term undrained-strength is used for the analyses. The higher short-term strength is likely to be available for the short term loading applied during an earthquake.

RECOMENDATIONS

1) A control-point survey-program could be conducted to monitor whether the site is still subject to down-slope movement, or consolidation. If long term monitoring found that movement was not happening the Factor of Safety could be adjusted to show at least 10% more stability, in my opinion.

2) Additional subsurface soil sampling and testing could provide a better basis for assessing slope stability. This would be an expensive program as the sampling and testing would have to be extensive and sophisticated.

3) A drainage trench could be installed to a depth of about 16 feet in the parking lot, as close to the building as feasible. The trench would angle toward the sewer manhole to a final depth 2 to 3 feet above the sewer pipe. The drainage trench should also extend to the south end of the building and would drain toward the sewer manhole with a depth of 10 to 12 feet. Deeper trenches could be constructed further increasing site stability, but would be more difficult to construct due to the location of the sewer pipe.

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August 24, 2006
2000 McGregor Drive

4) The existing retaining wall should be repaired where it has failed. Deeper piers, possibly with tie-backs should be installed at the location of the failed piers and probably extend at least a distance of two-piers on either side of the failed part of the wall. Stronger lagging should be also be installed. The remainder of the wall should probably be replaced during the next 5 to 10 years with deeper piers and tie-backs.

5) The parking area should be sealed and maintained to prevent water from infiltrating into the soil below.

6) Permeable surfaces elsewhere on the site should be covered with impermeable flatwork wherever possible.

7) Drainage should be improved and the water carried to a location near the creek where it will not erode the slope. Erosion control measures will be needed at the outlets of drainage pipes.

8) The slope below the retaining wall should be vegetated with Redwood trees or some other type of vegetation with extensive root systems and high evapo-transporation rates. If redwood trees are planted, they should be watered for several years until established and then pruned to maintain a maximum height of 10 to 15 feet.

9) If you require greater certainty for overall slope stability a system of deep piers extending 10 to 15 feet into bedrock with tie-backs could be installed. You will need to have access to the slopes below the retaining wall to construct an access road sufficiently wide to install the tie-backs. This will be an expensive repair. The cost will probably be in access of $300,000 and could much higher. The actual cost will depend on the final design for the wall, which will require further investigation to optimize the depth of embedment of the deep piers and to determine the depth of embedment of tie-backs.

<u>The above recommendations are general and not sufficient for construction or design. Please contact Terra Firma for specific recommendations.</u>

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LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report are based upon professional opinions about site conditions. For the purpose of preparing this report, the findings, and the recommendations it has been assumed that the soil conditions do not deviate from those identified during the subsurface investigation. If any variations or undesirable conditions are encountered in the future from that described in this report, our firm should be notified so that supplemental recommendations can be given.

2. This report is issued with the understanding that it is the responsibility of the owner, or his representative, to insure that the information and recommendations contained herein are called to the attention of the Architects and Engineers for the project and incorporated into the plans, and that the necessary steps are taken to insure that the Contractors and Subcontractors carry out such recommendations in the field.

3. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural process or the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated, wholly or partially, by changes outside of the control. This report should therefore be reviewed in light of future planned construction and then current applicable codes.

4. This report was prepared upon your request for our services in accordance with currently accepted standards of professional engineering practice. No warranty as to the contents of this report is intended, and none shall be inferred form the statements or opinions expressed.

5. The scope of our services was mutually agreed upon for this project. Terra Firma is not responsible if problems arise for conditions encountered that are not part of the scope of work for the project.

REGISTERED PROFESSIONAL ENGINEER
MARC J. RITSON
No. CE037100
Exp. 06 /2008
CIVIL ENGINEER
STATE OF CALIFORNIA

Marc Ritson
Registered Civil Engineer 37100

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Supplemental Soils Report

2000 McGregor Drive, Aptos.

March 5, 2007

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March 5, 2007
2000 McGregor Drive

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CALIFORNIA
COASTAL COMMISSION
CENTRAL COAST AREA

Joe Hanna, County Planning Geologist
Kent Edler, Geotechnical Associate
County of Sant Cruz Planning
701 Ocean Street, 4th floor
Santa Cruz, CA 95060

SUBJECT: APPLICATION 04-0650 (ZAR); GEOTECHNICAL REPORT ADDENDUM

Dear Mssrs. Hanna and Edler,

On behalf of my client, Randy Zar, I am submitting the geotechnical report addendum for the project at 2009 McGregor Drive, Aptos. The addendum supplements information contained in the original geotechnical report prepared for this project dated August 25, 2006. This addendum addresses the issues you both specified during our meeting January 3, 2007. More specifically, the addendum covers the following:

- The supplemental report addresses three topical areas: 1) the face of the slope 2) the body of the site, including a new retaining wall (or other slope stabilization measure) and site stability related to the building and 3) the building's foundation.
- The report determines if the building foundation needs to be augmented, and if so, what type of foundation retrofitting is necessary.
- Standard penetrometer testing (SPT) has been used to determine the stability of bedrock. It was agreed that a direct shear test is not needed.
- A single tri-axle test on one soil sample has been done. And this was done on the weakest of samples taken from new borings. All borings were drilled to at least 15 feet.
- The face of slope area has been addressed from an erosion control standpoint to prevent surficial erosion. Erosion control issues have been discussed on both the County-owned portion of the slope and the Zar-owned portion.
- Hand auguring at or near the toe of the slope (described in the above bulleted item) has been done to collect additional soil sample data for the slope area.

Please contact me at (831) 438-3216 if you need to discuss any of the items in the attached report.

Sincerely,

Marc Ritson, C.E.
Registered C. E. 37100

cc: Randy Zar
Kim Tschantz, Cypress Environmental
Randall Adams, County Planning

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March 5, 2007
2000 McGregor Drive

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TERRA FIRMA Engineering and Science

February 26, 2007
2000 McGregor Drive

Randy Zar
2000 McGregor Drive
Aptos, Ca, 95003

Dear Mr. Zar,

At your request, I prepared the following supplemental report for your project at 2000 McGregor Drive, Aptos. This supplemental report was prepared to respond to comments from the County of Santa Cruz Planning Commission, at their hearing on October 11, 2006. This report supplements the information provided in my Soils Report with Site Stability Analysis, dated August 25, 2006.

Introduction

Specifically, the supplemental report includes:

1) Recommendations for constructing a tieback, soldier-pile, retaining wall system to increase site stability. With the proposed retaining wall, the Factor of Safety (FoS) for the site (but not for surface slips down slope of the wall) is increased to 1.5 or greater.

2) Recommendations for building foundations, which can be used to limit total settlement of the building to less than 1 inch and differential settlement to less than a 1/2 inch.

3) Recommendations to improve site conditions to help maintain the portion of the site down slope of the proposed retaining wall.

Items #1 and #3 above address the subject parcel, APN 38-061-07, and the County "excess right-of-way" area adjoining the subject parcel.

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2.0 Scope of the Project

The project involves the reuse of an existing building for a new designated-use. It is assumed that only minor modifications to the existing building will be made. These modifications will add only minor new dead and live loads to the building. No new large fills will be placed at the site, except possibly adjacent to the proposed new wall. Larger new fills will be limited in extent and at least 20 feet from the building. Minor fills may be placed closer to the building to decrease the slope inclination immediately adjacent to the building.

Alternative locations and configurations for the proposed wall are possible. For example, moving the tie-back soldier-beam wall downhill would decrease the necessary depth of piers and tie-backs and could be cost beneficial, but would require County approval of a Riparian Exception. The scope of this report is limited to the wall location shown.

The investigative work and analyses done for the project show that the project is feasible. The recommendations in this report are not final construction-level-design recommendations.

3.0 Site of Description

As shown on Figure 1 and Plate PS-1, McGregor Drive is to the north of the site with Borregas Gulch located on the west and down slope from the developed portion of the site. The gulch is a riparian corridor with an intermittent stream that flows at an approximate right angle to McGregor Drive. A large commercial building, on relatively level grade, is located to the east. The southern end faces a residential parcel and the top-of-bank of Borregas Gulch.

The existing building is single story, about 100 feet long, and does not exceed 26 feet in width. As reported by the owner of the building, the building has existed in its present footprint since the 1960's, except that the southern-most approximately 20 feet of the building was added in the 1990's. The building has a slab foundation attached to perimeter footing, except at one location where a small part of the floor is cantilevered over the perimeter foundation.

The building is aligned approximately parallel to the axis of Borregas Gulch and is about 50-feet from the gulch's steep slopes at the northern end, and is at the top-of-bank at the southern end. The previous soil investigations found that the northern end of the building is likely to have been constructed on fill soils or soft native soils. At the southern end the building was placed on fill materials overlying native soils.

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In 1996 the parking lot, at the northern end of the site was extended toward Borregas Gulch as part of a project implemented by the County of Santa Cruz, Department of Public Works (County Sanitation District) to provide access to a buried sewer manhole. County Public Works prepared the project plan. The soils investigation for the 1996 grading was done by Reynolds and Associates (Reynolds) who also conducted construction oversight including conducting eleven field-compaction tests.

The grading work provided access to a sewer system constructed on the bank of Borregas Gulch (in the 1950.s), and was also provided the site with a widened parking area. As part of this project, fill soils were placed from McGregor Drive along the length of the building to a location about 70 feet along the building in a southerly direction. A retaining wall was constructed on the down slope side of the fill, for the length of the fill.

Reynolds reported (May 27, 1997); "As requested, we observed the base keyway and conducted testing services of the rough grading..." and "It is out opinion that the slope reconstruction has been adequately compacted and is completed." Reynolds did not conduct oversight or inspection for the retaining wall.

Near the southern end of the 1996 retaining wall, about a 12-foot length of the wall has failed. Based on field observation of the failed piers the embedment was inadequate, being only about 4 feet.

A surficial slip is located above Borregas Gulch near the outlet of the culvert under McGregor Drive, which is beyond both the project parcel and in the "excess right-of-way" area associated with this project. There may be another surface slip below the failed portion of the retaining wall. Other surface slips may be present, but due to the extensive vegetative cover on the slopes, visual evidence is not obvious.

4.0 Supplemental Field Investigation Conducted

In January of 2007, two supplemental borings were machine-augered at the top of the gulch-slope to identify the depth where soils are firm enough to provide embedment for piers and tie-backs. In addition, three shallow borings were hand augered on the slopes above Borregas Gulch, close to the creek, to estimate the dip of the bedding plane of the firm soil layer. Details of the Supplemental Investigation work are in Appendix 1.

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One boring was hand augered adjacent to the building foundation (under the deck) to collect a sample for settlement analysis.

The boring locations are shown on Plate PS-1. The numbering system for the borings has been revised from that shown in the August 25th report. Machine augered borings are now numbered consecutively from B1 to B5 (with the labels B1, B2 and B3 being the same in both reports). Numbering for Cone Penetrometer locations is unchanged. Hand augered borings are now identified as DCP-1 through DCP- 6.

Additional laboratory testing was done to a) refine the strength evaluation of the soils for slope stability analyses; and b) to evaluate the settlement potential of the site soils. Laboratory test data are in Appendix 2.

5.0 Findings from Supplemental Investigation

In general, the stratigraphy identified in the August 25th report was confirmed. As shown on Plates PS-2 and PS-3 (see Appendix 1 for boring logs), surface soils above the top of underlying firm-native soil (or bedrock), are comprised of lean clayey-sands to sandy-clays. Grain size analysis of these soils indicates that typically the percentage of sand-size grains (or larger) ranges from 45% to 55% with the soils having low to moderate plasticity:

Boring	B1	B2	B3	B4	FNDN	B5	
-Depth (ft)	5	17	17	12	4	19	
-Liquid Limits (%)	23	30	23	30	31	33	ave. = 28.3
-Plastic Limits (%)	17	16	16	18,	17	17	ave. = 16.8
-Plasticity Indices	6	14	7	12	14	16	ave. = 11.5

(see Appendix 2 for detailed data)

The supplemental investigation identified firm-soil (or bedrock) at locations B4, and DCP 3, 4 and 6, as shown on Plates PS-2 and PS-3. Firm soil was identified at location B5, but due to the limitations of the portable drill-rig, the boring was terminated at depth of 28 feet. The portable drill rig had to be used, as the adjacent property owner did not grant permission to access the drilling location with a truck-mounted rig.

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Firm-soil (or bedrock) was found 5 to 6 feet below ground surface just above the creek banks at the bottom of the slope. Interpreted depth (data from CPT-1, CPT-2, B4, and DCP-3) indicate that firm-soil (or bedrock) is about 25 feet below the base of the existing retaining wall along Section B-B' (see Plate PS-2), and 25 feet below the base of the proposed retaining wall along Section E-E' (see plate PS-3).

Two laboratory consolidation tests were completed (see Appendix 2 –Laboratory Data and Appendix 3 –Consolidation Settlement Analysis). Samples tested were:
a) A clayey sand soil from under the parking lot (B4 depth 14 feet); and
b) A clayey sand from DCP-5 (depth 4 feet), adjacent to and below the depth of the existing building foundation.

The existing large fill, at the site, was constructed in 1996. Based on the testing done, the calculated total consolidation settlement of the parking lot area (due to placement of the 1996 fill) is 3 to 4 inches (see Appendix 3, Figure 5). The consolidation tests and analyses show that 90% of the expected settlement would occur in less than 4 1/2 years for a 12-foot vertical drainage path (see Appendix 3, Figure 6). For the soil profile at the site, a 12-foot drainage path would be a worst-case scenario. As the parking lot fill was constructed 10-years ago, no further significant consolidation-settlement should be expected in the area of the fill.

For narrower, spread-footing, building foundations, the calculated consolidation settlement for new loads on a 1.5-foot-wide footing is 0.1 inches per 100 pounds per square foot (psf) (see Appendix 3, Figure 5). The drainage path is much shorter for the building footings and the time to consolidation is less than a 1/2-year (see Appendix 2 and Appendix 3, Figure 7). Unless new loads have been added to the building in the last year, or will be added in the future, no significant new settlement should occur under the building at this time.

Laboratory strength testing of site soils was also done. A sample from B5 was subject to a Staged Triaxial test, and three samples from B4 were subject to Unconfined Unconsolidated Compression tests (see Appendix 2).

These data were used in the slope stability analyses conducted, as described in the next section of this report. These data are also used for assessing appropriate building foundation bearing capacity recommendations. Also, the unconfined-compression-test data validate data from the Cone Penetrometer soundings done during the 1st investigation.

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To determine if settlement of the parking lot area is abnormal, cross-sections were made of the site parking lot and across the width of McGregor Drive. The cross-section for McGregor was done approximately over the thickest part of the McGregor Drive fill; which is similar in height and adjacent slopes to the site parking lot area. The data indicates that there is little difference in the slopes across both of the pavement widths. Both areas show cracking which is likely to have been caused by settlement, but there appears to be nothing particularly abnormal about the settlement of the site parking area, based on the compared cross-sections. The cross-sections are shown on Figure 2.

6.0 Proposed Tie Back Retaining Wall

The proposed tie back retaining wall is shown in plan view on Plate PS-1, and in section on Plates PS-2 and PS-3. The system includes a six-foot high retaining wall at the top, with 25-foot to 35-foot long, 2-foot diameter soldier-beam-piers, spaced 10-feet on center, and with a tie-back at each pier.

7.0 Soil Strength and Water Table for Slope Stability Analyses

Based on the site investigations conducted, the stability of the site slopes is very dependant on the interaction between subsurface water and the site soils. Unsaturated, but wet, site soils have considerably more strength. Saturated soils are weaker. To identify whether saturated or unsaturated soil strengths should be used for the stability analyses, data from the field and laboratory investigation were compared.

The site is located in an area where there are no large catchments for precipitation and therefore the potential for large accumulations of groundwater under the site is limited. The site slopes incline from 40° to 50° degrees; horizontal to vertical ratios of 0.8 : 1 to 1.2 : 1. Due to the presence of the steep slopes, it is not likely that groundwater can be very elevated at the site as the steep slopes form a free surface for any accumulated groundwater to drain through.

The investigative work done at the site supports the above conclusion. The 2006 fieldwork was done after very heavy rains in March 2006, during which there was about 40 consecutive days with rainfall. This very extended period of rainfall caused numerous land slippages (some very large) throughout the County. When the 2006 site field borings were made, water was found in boring B1 to extend from a depth of 21-feet to the bottom of the boring at 26-feet. The water

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depth in B1 extends over the depths of the firmer underlying soils and not into the softer soils above. No water was found at boring B2, with B2 having a depth of 27-feet. Water depth was not measured in B3. Borings B4 and B5 did not find water, but were drilled in January of 2007, when little rainfall had occurred and therefore water data from these borings are inconclusive.

As the knowledge about groundwater is inconclusive, the stability analyses conducted for this report assume there is, potentially, a water table that starts about 4 feet above the top of the firm soil layer (or bedrock) and is inclined parallel to the firm soil layer, which drains at the bottom of the slope into the creek.

Slope-stability-model assumptions are based on the assumed groundwater condition stated above. The soils above the water table were assigned strengths that are typical for unsaturated soils found at the site. These strengths are based on the CPT data and laboratory test data for unsaturated samples. These soils are typically clayey-sands to sandy-clays and are likely to be fill-materials at the top of the soil profile, and weathered old-top-soils or colluvium in the lower part of the profile. These soils were assigned a friction angle (Fi) of 31 and cohesion (C) of 250 psf for the top-most soil and Fi = 28 degrees C of 250 below.

Assumed to be saturated is about a 4-foot thickness of soil (above the interface between upper-softer-soils and firmer underlying soils). This 4-foot layer is affected by groundwater in two different ways. The soil in the top portion of the 4-foot thickness (about a 2 foot thickness) is assumed to have strength that is best estimated from Total Stress tests. Total Stress analysis is based on water not being able to migrate from the soil when it is loaded. As the soils above and below this zone are relatively impermeable, the use of Total Stress strengths appears appropriate for this zone.

The Total Stress characteristics of the soil where measured using a sandy-clay sample from the interface area. The sample used was intentionally selected to be relatively weak, based on its Standard Penetration Test (SPT) blow counts. Based on visual observation of the sample and laboratory testing, the sample was typical of soils found at the base of the weaker upper soils. The sample was subjected to a staged triaxial test, consolidated, undrained, and with pore pressure measurements (see Appendix 2 for test results). The Total Stress friction angle (Fi) is 19 degrees and the cohesion (C) is 130 pounds per square foot (psf). This soil is the weakest in the slope stability models.

Soils within the interface, but below the soils described above, are assumed to be the same material but are also assumed to drain through the underlying more sandy soils. The strength assigned to these soils is based on based on Effective Stress testing with Fi =31 degrees C=100 psf.

The Cone Penetrometer soundings data for CPT-2, 3, 4 and 6 are consistent with the assumed profile described above. The data for CPT-1 and CPT-5 show negative pore pressure in the weak soil zone above the interface; logs for CPT-1 and CPT-5 are in Appendix 1. The negative pore pressures indicate that the soil strength increases under loads rather than decreasing as described above. At these two locations the soils are stronger then those described by the assumed soil profile. The site slope stability is therefore probably greater than that used in this report for the existing site and for the site with the proposed retaining wall system.

Table 1 below summarizes the available strength data for the site.

Saturated Drained		Direct-Shear Drained		
Boring	**Depth, ft.**	Fi	C	
B1	5	39	230	
B2	16	35	0 to 150	
		TX CD w/PP		
B5		31	100	**SATURATED**
Saturated Undrained				
		TX CD w/PP		
B5		19	130	
		Direct-Shear Undrained		
B3	17	22	300	

In-Situ CPT	**Depth, ft.**	Fi	C	
CPT.s 1 to 6		not calculated for cohesive soils	350 to 4500	
				UNSATURATED
In-Situ	Unconsolidated-Unconfined Compression			
B4	10	0	490	
B4	14	0	578	
B4	20	0	2757	

The computer program GeoStru was used for slope-stability analyses done in this report. As described in the first report the program WinStabl (University of Wisconsin) was used to verify

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GeoStru results. The results of the slope stability analyses for the existing site, using the data above, for Sections B-B' and E-E' are shown on Figures 3 and 4.

Comparative analyses were also made assuming all the soil in the interface zone is: a) the lower strength soil; or b) the higher strength soil. If all the soil is weak, the FoS decreases about 7% compared to the two-soil condition. If all the soil in the interface zone is assigned the higher values, the FoS increases about 28% compared to the two-soil condition. The assumed condition of a mixed interface of weak soils appears to be a reasonable but conservative scenario for the site, given the available information. The results of the comparative analyses are shown on Figures 5 and 6.

Given the long-term historical stability of the site, assuming conditions that lead to a FOS of less than 1 for deep-seated slides at the existing site, without seismic loading, is not rationale.

The existing slopes, down-slope of the existing-retaining-wall, have exhibited surficial instability at one location, and perhaps at others. Instability indicates a FoS less than 1. The surface slip(s) is/are likely to be caused by saturation of the surface soils during periods of extended precipitation, or undercutting of the slopes by erosion. The surface slip(s) are not relatable to instability for deeper slips, the deeper slips being a different problem. Surface slips along creek banks are not abnormal and are a part of the natural evolution of gulches and creeks.

8.0 Slope Stability Analyses and Stability Due to Seismic Loads

The site is located in a seismically active area. The effects of seismic activity on the site slopes are difficult to predict, as there is little coherent knowledge about the effects of seismic forces on cohesive soils.

Consolidation of saturated clayey soil causes excess pore pressures in the soil. During the consolidation period, a seismic event would further increase the internal water pressure and decrease slope stability. However, based on the consolidation tests and analyses done, consolidation is complete at the site and excess pore pressures are not likely to occur. Unsaturated cohesive soils should be expected to increase in strength during a seismic event. The increase is due to the tendency of soils to expand under short-term load. Soil expansion causes increased capillary tension in fine pores, which are intrinsically a part of a clay soil structure. This strength increase can be significant, adding 5% or more to the strength of the soil.

The affects of seismic loads on soils in the saturated zone are unpredictable.

Due to the uncertainty about the effects of a seismic event on the strength of site soils, but given that the majority (80% to 85%) of the site soils (above the firm underlying soils) are unsaturated, it is assumed that strength increases more than balance strength decreases. A 10% net increase in soil strength due to seismic loading is used in the analyses.

To evaluate site slope-stability under seismic loads, an assumed value for horizontal and vertical acceleration was estimated. The assumed horizontal acceleration increases the driving force downhill and the assumed vertical acceleration decreases the frictional forces at the interface of the slip plane, also increasing the driving forces. Soils typically attenuate seismic forces and a reduction factor can therefore be applied to the expected peak-seismic-acceleration. For the site, the computer program, GeoStru, estimated horizontal acceleration to be 0.21 times the acceleration due to gravity with the vertical acceleration being 1/2 the horizontal.

The computer model was used to determine the necessary capacities for the structural elements of the tie-back retaining wall which would provide a FoS of 1.2 for seismic loads, as described above. The results of the analyses are shown on Figures 9 and 10. Recommendations for structural elements of the wall system are in Section 10 of this report.

9.0 Comparative Retaining Wall Analyses

The computer program Shoring Suite V8 (CivilTech Software) was used to compare the GeoStru slope stability analyses with an alternative method. Shoring Suite V8 uses analysis methods intended for design of retaining structures for cuts and fills. The methods used in the model are based on those developed by the United States Department of the Navy, other federal agencies, and other recognized entities.

The model input into the program is a 10 foot high wall with a 45° degree down-slope slope starting at the base. As only a 6 foot high wall is proposed, the model is forced to assume that the 4 feet of soil below the base of the proposed top-retaining-wall does not provide any resisting strength. In addition, all the soils down-slope of the wall will also have less strength. Soil strengths used in the model were determined from correlations to field standard penetration test blow-counts and comparison to test data. The data was entered into the program for soils

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with both cohesion and friction. But, to use the model for seismic forces an 'equivalent soil' with only frictional strength (rather than both cohesion and friction) is calculated.

The results of this model show required strength of the wall structural elements to be about 2/3.s or less than that calculated by the Geostru model. The results are shown in in Appendix 4. One large difference between the models is that the GeoStru model includes soil that extends farther back (upslope) from the wall than the Shoring Suite model. The GeoStru results are used to provide recommendations in this report.

10.0 Recommendation For Tie Back Retaining Wall

The proposed tie-back retaining-wall, shown on plates PS-2 and PS-3, is feasible. The design parameters, described below, are for the wall shown. These recommendations are not sufficient for actual construction. Also, other configurations of the wall are possible, but alternative recommendations will have to be prepared for different configurations.

The wall structural elements include:

1) Up to a 6-foot high retaining wall may be placed above the level of the tie-backs.

2) Piers with a minimum 2-foot diameter, at a maximum spacing of 10-feet on center, are embedded 12-feet into firm underlying soil.

3) Tiebacks are also at 10 feet on center, and embedded into the underlying firm soil layer 8-feet to 12 1/2 feet.

The up-to-6-foot high wall (at the top of the tie-back retaining-wall system) may be designed using an active equivalent hydrostatic pressure of 50 psf (zero psf at the top, increasing at 50 psf per foot of depth). The design seismic load is 8 x H^2 (H= height of wall) applied at a point 0.6 H above the base of the wall. The retaining wall will have to be designed to transfer loads to the tie-backs and piers below. If vehicles with wheel loads greater than 1 tons are to parked closer than 4-feet to the wall, additional loads will need to be applied to the wall.

The piers below the upper retaining wall should be designed for a bending capacity of 72 Kip-feet, with typically a 25-foot length from the bottom of the upper retaining wall to the top of underlying firm soil. The minimum embedment of piers into the firm underlying soil is 12 feet.

Each pier should have a tie-back. The center-to-center spacing is 10-feet. The required tie-back capacity is 50 kips for the portion of the wall extending from the north end of the parking lot to the 15 feet south of Section B-B'. From there to the end of the wall the required tie-back capacity is 80 kips. The allowable transfer capacity between the tie back and firm native soil is 6 kips per foot for 6 to 8 inch diameter, low-pressure-grouted tie-backs, based on the Federal Highway Administration Publication FHWA DP-68—1, 'Permanent Ground Anchors', March 1984, page 24. This yields a minimum 8 to 12.5-foot grouted lengths into the firm underlying soil, depending on the location along the wall. However, the actual embedment length must be determined in conjunction with the manufacturer and installer of a specific tie-back system. Many proprietary systems have higher transfer capacity. The manufacturer and installer of the tie back system should be contacted to provide design capacities for their systems. All tie-backs should be tested after installation to verify adequate capacity.

Based on the analyses, with seismic loads, the soldier beams piers will require a bending capacity of 250 Kip-feet for the portion of the wall extending from the north end of the parking lot to the 20 feet south of Section B-B' with tie-backs having a capacity of 180 kips each. From there, to the end of the wall the soldier beams piers required a bending capacity of 180 Kip-feet required tie-back capacity of 185 kips. Applicable, code allowed, load-combination reductions or increases must be applied to the above requirements. Reductions or increases in materials strengths are also applicable. Soil strength may be increased by 1/3 for tie-back load-transfer and for soldier-beam-pier embedment. Seismic loads for factoring may be calculated by subtracting non-seismic from seismic requirements to derive seismic increase.

11.0 Building Foundation Bearing Capacity

Based on the strength testing done for samples collected at the site, the site soils have adequate capacity to support 633 psf with a maximum allowable total load of 950 pounds per lineal foot (plf), if the slopes below the foundations are reduced to an inclination of 2:1 (horizontal to vertical), with the face of the bottom of the footing being offset horizontally 5 feet from the face of the slope. At the southern end of the building, the slopes below the foundations will have to be filled to attain an inclination of 2:1. Infilling may necessitate the construction of short retaining walls, or may require using the proposed tieback wall as support for the new slope.

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The bearing capacity of perimeter footings may be increased if a floor slab is attached to the perimeter footing. The increase is the allowable shear capacity between the slab and the footing up to a value of 400 psf.

A settlement of 0.1 inches per 100 psf of new load is expected. Differential settlement should be limited by not placing new loads in a manner that causes differential settlement to exceed prescribed limits. It should be assumed that even with careful planning of the foundation system some differential settlement will occur, that will not substantially affect structural integrity, but may cause cosmetic cracking of slabs, tiles, plaster or stucco.

12.0 Down Slope Recommendations

The slopes below the proposed tie-back retaining wall cannot be easily improved using geotechnical approaches without causing major disturbance to the slopes. Due to the gradient of the slope and the fact that it is within a sensitive habitat (riparian corridor), it is recommended that that a botanical approach be employed to improve the stability of this portion of the site.

The subject slope, in some areas, lacks the typical tree and shrub cover found in most riparian habitats. This has made the slope more susceptible to surficial erosion than if the woody vegetation had been retained.

Along the creek bank immediately adjacent to the creek erosion is occurring which can undercut the banks and lead to surficial slope failures farther upslope. One such slip has occurred near the headwall for the outlet of the culvert under McGregor Drive (which is beyond the subject parcel and "excess right-of-way" area associated with the project). It is recommended that the toe of the slope be stabilized by biotechnical buttressing using fascines or by rock buttressing. The biotechnical approach is the superior of the two alternatives, as discussed below.

In general, vegetation has a beneficial effect on slope stability by the processes of interception of rainfall, and transpiration of groundwater, thus maintaining drier soils and enabling some reduction in potential peak groundwater pressures. Vegetation roots reinforce the soil, increasing soil shear strength while tree roots may anchor into firm strata, providing support to the upslope soil mantle through buttressing and arching. A small reduction in soil moisture induced by the roots can substantially increase cohesion and can have a major effect on reducing shallow slides.

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The mechanical effect of vegetation planting is not significant for deeper-seated landslides, while the hydrological effect is beneficial for both shallow and deep landslides. However, vegetation may not always assist slope stability. Destabilizing forces may be generated by the weight of the vegetation acting as a surcharge and by wind forces on the vegetation exposed, though both these are very minor effects. Roots of vegetation may also act adversely by penetrating and dilating the joints of widely
jointed rocks.

Fascines are live branch cuttings, usually willows, bound together into long ~~tubular~~ bundles used to create a "geo-berm" to stabilize slopes and stream banks. This biotechnical approach to stabilizing the toe of the slope is less invasive than using rip rap. Wedge-shaped stakes are installed vertically into the treated areas to increase the stability of the willow bundles until they root. When the live willow branches root and sprout they provide long-term soil reinforcement. Stems, rope ties and wedge-shaped wooden stakes all combine to provide temporary structural reinforcement. Minor amounts of fill material may be needed to install the fascines in stepped back fashion. This alternative is recommended as it would result in less construction impacts to the slope than the rip-rap approach. An erosion control or botanical consultant should be retained to evaluate this alternative towards providing fascine construction and installation details. This should be part of a comprehensive erosion control plan for the entire slope.

The rock buttressing method would require the placement of rip-rap (large boulders, 3 to 4 feet in diameter) at the toe of the slope to reduce further slippage. An excavation will have to be cut into the bank to place the rocks. The excavation should be lined with a heavy-duty filter fabric prior to placing the rocks. Smaller rocks should be placed into the voids of the larger rocks to help lock the structure together and reduce voids spaces for upslope soils to move into.

13.0 General Recommendations

The site drainage should be improved to minimize water infiltrating into the site (either from irrigation or precipitation). Water captured by the drainage system should be transported down to the creek bed in enclosed pipes that are secured to the slope surface. Flexible plastic pipe, 6 to 12 inches in diameter, is recommended. The outlet of each pipe should discharge onto an energy dissipater. The energy dissipaters should also be secured to the ground surface to prevent movement. No trenching of the slope should occur when placing the drainage pipes on the slope.

14.0 Conclusions

The analyses done in this report are based on conservative assumptions that probably underestimate site soil strength. These analyses indicate that the site can be improved to meet County requirements for slope stability. The analyses also show that little or no new settlement due to consolidation of soils underlying the parking lot and building are expected if no new loads are placed. The slopes down-slope of the proposed retaining wall should be improved using biotechnical or mechanical means, or a combination of both. Site drainage should be carefully controlled to minimize infiltration of water into the site.

15.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report are based upon professional opinions about site conditions. For the purpose of preparing this report, the findings, and the recommendations, it has been assumed that the soil conditions do not deviate from those identified during the subsurface investigation. If any variations or undesirable conditions are encountered in the future from that described in this report, our firm should be notified so that supplemental recommendations can be given.

2. This report is issued with the understanding that it is the responsibility of the owner, or his representative, to insure that the information and recommendations contained herein are called to the attention of the Architects and Engineers for the project and incorporated into the plans, and that the necessary steps are taken to insure that the Contractors and Subcontractors carry out such recommendations in the field.

3. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural process or the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated, wholly or partially, by changes outside of the control. This report should therefore be reviewed in light of future planned construction and then current applicable codes.

4. This report was prepared upon your request for our services in accordance with currently accepted standards of professional engineering practice. No warranty as to the contents of this report is intended, and none shall be inferred form the statements or opinions expressed.

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5. The scope of our services was mutually agreed upon for this project. Terra Firma is not responsible if problems arise for conditions encountered that are not part of the scope of work for the project.

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