Charles Holman
901 Etheldore Street
Moss Beach, Ca. 93038
charlie@charlesholman.com
650-747-0769

November 28, 2016

Bernadette Mahoney and Kelly Hirano
11406 Lindy Place
Cupertino, Ca. 94025

Re: Hillside Exception Application @11406 Lindy Place

Dear Neighbor-

The Mahoney/Hirano Family wishes to construct a new pool and patio, along with a new deck detached pool house at their residence. We have submitted plans (enclosed) to the City and the Planning Department has suggested we reach out to our immediate neighbors and to get your feed back/approval for the project.

As you can see from the enclosed plan, the pool has been relocated and a new deck is attached to the existing home connected to the proposed pool house. No other changes are proposed to the main residence. We believe location of the new pool house and adjacent patio will not create any invasion of privacy issues as far as our neighbors are concerned.

We would welcome your comments regarding this project. Please let us know. If it meets with your approval we would appreciate your signing off that you have no objection to the proposed plans and returning. These letters will accompany our other application materials when we make our presentation to the Planning Commission.

Please let us know if you have any questions or comments and we very much appreciate you support.

Charles Holman Design Associates
650-747-0769
Charlie@charlesholman.com

kelly_hirano@yahoo.com

If you have no comments, and the project meets with your approval, we would very much appreciate that you sign the attached copy of this letter and return it to us in the stamped return envelope.

I, ____

Neighbor at address______________________________

Approve of the project as drawn.

Signed____Date________

Charles Holman
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Charles Holman Design Associates
650-747-0769
Charlie@charlesholman.com

kelly_hirano@yahoo.com

If you have no comments, and the project meets with your approval, we would very much appreciate that you sign the attached copy of this letter and return it to us in the stamped return envelope.

I, JAMES MOORE

Neighbor at address 21962 LINDY LANE

Approve of the project as drawn.

Signed James A. Moore Date 12/10/2016

From:	Beth Ebben
To:	Catarina Kidd
Subject:	FW: Strong opposition to request for hillside exception for pool, patio, and pool house at 11406 Lindy Lane [Charles Holman/Mahoney residence] (APN# 356-24-011/Application No. EXC-2016-08).
Date:	Monday, March 13, 2017 8:43:00 PM

From the Planning Department's general mailbox:

From: sara arzeno [mailto:s.arzeno@gmail.com]
Sent: Monday, March 13, 2017 11:05 AM
To: City of Cupertino Planning Dept.
Subject: Strong opposition to request for hillside exception for pool, patio, and pool house at 11406 Lindy Lane [Charles Holman/Mahoney residence] (APN# 356-24-011/Application No. EXC-2016-08).

planning@cupertino.org

Please confirm receipt of this message. :-)

To the City of Cupertino Planning Department,

We are writing to express our strong opposition to the request for hillside exception to build pool, patio, and pool house at 11406 Lindy Lane [Charles Holman/Mahoney residence] (APN# 356-24-011/Application No. EXC-2016-08).

Lindy Lane history includes at least one catastrophic hillside at 21852 (James residence) subsequent to construction of pool etc. on the hill.

Zoning/building laws for hillsides/slopes have been designed and enacted precisely to avoid future damage and destruction to the hillside communities. There is no justification to provide exceptions that would endanger the hillside residents and their homes, especially in an area with a demonstrated precedent for devastating slides.

We imagine city engineers will be examining the potentially destructive environmental impact of this request and city planners will be evaluating the unreasonable risk and potential dangerous consequences associated with such a request.

Living in a hillside community requires respect for the environmentally sensitive geography and for the community and residents. It is hard to imagine that the desire to build a pool and outbuildings could possibly trump the safety of an entire community of residents.

We appreciate the opportunity to voice our concerns and opposition.

Kindly confirm of receipt of this message.

Regards,

Sara Arzeno

21902 lindy lane

Cupertino, CA

From: Beth Ebben
To: Catarina Kidd
Subject: FW: (APN# 356-24-011/Application No. EXC-2016-08) - Please Confirm Receipt
Date: Tuesday, March 14, 2017 1:41:26 PM

From the Planning Department's general mailbox:

From: Jonathan Arzeno [mailto:jonatharz@gmail.com]
Sent: Monday, March 13, 2017 8:55 PM
To: City of Cupertino Planning Dept. <planning@cupertino.org>
Subject: (APN# 356-24-011/Application No. EXC-2016-08) - Please Confirm Receipt

To Whom it May Concern,

My name is Jonathan Arzeno and I am a lifelong resident of Cupertino, and **I am writing to express my strong opposition to the request for hillside exception to build pool, patio, and pool house at 11406 Lindy Lane (APN# 356-24-011/Application No. EXC-2016-08)**. I feel lucky to have the privilege to live in this city for the past 22 years and am proud to live in such a beautiful area. One thing that I have always cherished about Cupertino is the beautiful natural environment that surrounds our community.

As a student at Regnart Elementary, Kennedy Middle School, and Monta Vista High School, I have extremely fond memories of looking out to the surrounding beautiful and open hills cape that surrounded me with joy. The open hills behind my house, at 21902 Lindy Lane, provided a natural escape during my childhood and an area in which I could enjoy just being a kid in a beautiful and natural environment. I have spent countless hours exploring the open areas that surround and define Cupertino.

Recently, I have become increasingly concerned by the aggressive onslaught of development in Cupertino. It seems that each time I walk through Fremont Older Open Space, a newly constructed house dots the landscape - slowly but surely developers are encroaching on the land that the City once promised the residents of Cupertino would be theirs to enjoy as nature forever.

The question I ask, is when will the City of Cupertino stand up for what is in the best interest of the residents of our community rather than caving in to the demands of developers? I ask the City of Cupertino to please think about the irreversible effects of such aggressive and untamed development, and in this case, especially the dangers of overlooking established regulations in a hilly and earthquake-prone residential community. This regulations exist for a reason, and I urge you not to overlook their value. The potential for both personal and property damage that this specific request could bring to many residents brings far outweighs the right that one resident has to build a pool in an area the city has already deemed unfit for such development. The owner of that house, bought the property knowing the regulations, and cannot simply change the rules of the game to fit their own needs.

The decision here should be fairly simple. Cupertino has for too long caved into developers rather than acting in the best interest of residents. Please, protect the beautiful community that we live in as well as the safety of the residents of my neighborhood. Do not allow this development. I cannot attend the meeting on this due to the fact that I will be at work, but please feel free to reach out to me with any questions. I strongly oppose this development effort, and again urge you to act in favor of the majority rather than yet another resident who wishes to hurdle city regulations in pursuit of reckless development.

Best,

Jonathan Arzeno

From: Catarina Kidd
To: "sara arzeno"
Subject: RE: WRONG ADDRESS? Strong opposition to request for hillside exception for pool, patio, and pool house at 11406 Lindy Lane [Charles Holman/Mahoney residence] (APN# 356-24-011/Application No. EXC-2016-08).
Date: Monday, March 20, 2017 10:31:00 AM

Sara,
Please add to your contact information my direct email: catarinak@cupertino.org

Jeff told me your message from Friday.

As advised in my earlier email to you, the owner is looking to revise plans and therefore it will be re-noticed in any case.

We have noted the error of "Lane" rather than "Place", so it will be corrected as well.

Please call me directly if there are any questions.

Sincerely,

Catarina S. Kidd, AICP, Senior Planner
City of Cupertino | Community Development
10300 Torre Avenue, Cupertino, CA 95014
408-777-3214 | catarinak@cupertino.org

From: sara arzeno [mailto:s.arzeno@gmail.com]
Sent: Monday, March 20, 2017 9:06 AM
To: City of Cupertino Planning Dept. <planning@cupertino.org>
Subject: WRONG ADDRESS? Strong opposition to request for hillside exception for pool, patio, and pool house at 11406 Lindy Lane [Charles Holman/Mahoney residence] (APN# 356-24-011/Application No. EXC-2016-08).

Hello again Catarina,

I spoke with Jeff/Geoff(?) on Friday to confirm that there appears to be an error on the communication sent out regarding the 30% hisslide exception. He told me he would email you and cc me - but I do not see an email from him,

To recap the conversation briefly, the address the city used on the communication to all potentially affected residents is incorrect - the message indicates the property is on Lindy Lane (no such address on Lindy Lane) - which is why so many of us were confused. We believe the address should be Lindy Place which is very different and on a very steep hillside above those of us living on Lindy Lane.
We imagine that the city will therefore need to re-send all the communications with the correct information and re-start the process to allow folks a chance to respond.

Please confirm receipt of this message and let us know what the next steps will be.

Many thanks and kind regards,

Sara

On Mon, Mar 13, 2017 at 11:05 AM, sara arzeno <s.arzeno@gmail.com> wrote:

> planning@cupertino.org
>
> Please confirm receipt of this message. :-)
>
> To the City of Cupertino Planning Department,
>
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>
> We appreciate the opportunity to voice our concerns and opposition.
>
> Kindly confirm of receipt of this message.
>
> Regards,
>
> Sara Arzeno
>
> 21902 lindy lane
>
> Cupertino, CA

From:	Beth Ebben
To:	Catarina Kidd
Subject:	FW: Opposition to hillside exception to build pool etc.
Date:	Monday, March 13, 2017 8:40:12 PM

From the Planning Department's general mailbox:

From: Mohammed Hossain [mailto:sharminsalim2@gmail.com]
Sent: Monday, March 13, 2017 7:26 PM
To: City of Cupertino Planning Dept.
Subject: Opposition to hillside exception to build pool etc.

To the City of Cupertino Planning Department,

We are writing to express our strong opposition to the request for hillside exception to build pool, patio, and pool house at 11406 Lindy Lane [Charles Holman/Mahoney residence] (APN# 356-24-011/Application No. EXC-2016-08).

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Kindly confirm of receipt of this message.

Regards,

Mohammed Hossain

Sharmin Hossain

21882 Lindy Lane

Cupertino, CA

From: Larry Wilson
To: Catarina Kidd
Cc: Larry Wilson
Subject: Re: 11406 Lindy Place
Date: Monday, May 15, 2017 1:23:21 PM
Attachments: ATT00001.htm
Teledyne Eng Svcs Tech Rpt 10427-1.pdf

Hi Catarina,

I am attaching the Rhodes and Purcell soil report which we mentioned earlier. I was unhappy to note that Murray did not reference the Rhodes and Purcell report in their report since I had shared a copy of the Rhodes and Purcell report with Murray when they were doing their field tests.

Please make this document available along with the other documents relating to this matter.

Larry

On Apr 27, 2017, at 7:56 AM, Catarina Kidd <CatarinaK@cupertino.org> wrote:

Larry,
We only post materials for each homeowner's specific application/location, but if you want to provide a public comment/letter and attach the report as part of your letter, I can add it that way.

Sincerely,

Catarina S. Kidd, AICP, Senior Planner
City of Cupertino | Community Development
10300 Torre Avenue, Cupertino, CA 95014
408-777-3214 | catarinak@cupertino.org

From: Larry Wilson [mailto:dreamproperties@icloud.com]
Sent: Wednesday, April 26, 2017 3:46 PM
To: Catarina Kidd <CatarinaK@cupertino.org>
Cc: Larry Wilson <dreamproperties@icloud.com>
Subject: 11406 Lindy Place

SEPARATE ATTACHMENT

TO

TELEDYNE ENGINEERING SERVICES

TECHNICAL REPORT 10427-1

SEPTEMBER 26, 1983

Purcell, Rhoades & Associates

Consultants in the Applied Earth Sciences

SLIDE REPAIR
YOUNGER'S RESIDENCE AND CONTIGUOUS PROPERTIES
2186 LINDY LANE
CUPERTINO, CALIFORNIA

FOR

TELEDYNE ENGINEERING SERVICES

Purcell, Rhoades & Associates

Don Nebbt

Purcell, Rhoades & Associates

Consultants in the Applied Earth Sciences

~~780 E. Trimble Road, Suite 102~~
~~San Jose, CA 95131~~ Hayward
(408) ~~[illegible]~~
510-732-9890
Please Reply to This Office ☑

2504 Technology 94545

925 932 1177

3200-A Buskirk Avenue
Walnut Creek, CA 94596
(415) 932-1177
☐ Please Reply to This Office

Project No. 3179
September 15, 1983

Teledyne Engineering Services
3938 Trust Way
Hayward, CA 94545

Attention: Mr. Tom Adams

SUBJECT: Slide Repair, Younger's Residence and Contiguous Properties, 21862 Lindy Lane, Cupertino, California

Gentlemen:

At your request, we have performed a geotechnical slide study for the subject property. Our study has included alternate repair considerations for the correction of the slide at the rear of the Younger residence with the possibility of extending the repair to the adjacent contiguous properties. We conclude from our study that provided the grading and drainage and/or retaining structures are constructed in conformance to the recommendations of this report, the risk of further landsliding will be mitigated to an acceptable hazard.

Our recommendations are presented in the following text of this report and include the alternates to be selected based upon economic considerations tempered by time constraints and restrictions of working with the adjacent property owners. The final plan selected must be reviewed prior to contract by this office to clarify construction sequence and methods in view of the precarious condition of the slide headscarp. In addition, governmental agencies represented by the City of Cupertino must review plans and calculations for the building permit.

We refer you to the text of this report for detailed recommendations. If you have any questions or desire additional information, please contact the undersigned.

Very truly yours,

PURCELL, RHOADES & ASSOCIATES

Irving D. Affeldt

Irving D. Affeldt, C.E.G.
Associate

Daniel J. Rhoades

Daniel J. Rhoades, C.E.
Principal

pl

REPORT

REPORT

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PURPOSE

In accordance with the request of Mr. Tom Adams of Teledyne Engineering Services, a detailed geotechnical study and evaluation of the existing slide conditions to the rear and adjacent to the Younger residence (Lot 28) was initiated on August 4, 1983. The results of our evaluations, interpretations and analyses were utilized in the formation of the soil engineering recommendations included within this report. The purpose of this study was to provide geotechnical planning and design criteria for the installation of walls and regrading of the slide damage to the rear of the Younger property.

SCOPE

The initial portion of our study consisted of review of available published and unpublished geotechnical information relevant to the property. Subsequent to multiple site reconnaissances, the subsurface conditions at the property were evaluated by the advancement and sampling of seven test borings down the length of the main slide to the rear of the Younger residence. Appropriate testing of the obtained samples was performed in our laboratory, and an in-depth analysis of the test results was accomplished. The results of this work is included in the following text, tables and figures.

Prior geotechnical studies performed during development of the Candy Rock Subdivision have been reviewed with relevant portions incorporated into this study. In addition, a report prepared by Cleary Consultants dated October, 1982 for the main slide mass that occurred during the 1981-1982 rain season was reviewed. This study was useful because of the subsurface information obtained in the slide mass prior to further movement during 1982-1983, which resulted in a new slide occurring within the toe of the old slide mass. In addition, our files were reviewed for all recent pertinent information including aerial photograph interpretation of the conditions within the hillside.

LOCATION AND DESCRIPTION

The property is located in Santa Clara County, on a north-facing slope in the eastern foothill region of the Santa Cruz Mountains (see Figure 1). The ground surface to the rear of the Younger residence where the slide occurred ascends moderately toward Lindy Drive. It is locally steeply sloping, and in places hummocky, terraced and disrupted. The general area and the parcel under study has been extensively graded and filled during past development of the surrounding properties.

The original soils investigations for this subdivision were performed in November, 1962 and May, 1965 by Gribaldo, Jacobs and Jones (GJJ). The reports described the original terrain as consisting of rounded hills with steep topography in the southern and northwest portions of the property. A site plan showing the original contours is presented as Figure 2. For comparison, Figure 3 shows the present site conditions. Subdivision development entailed extensive grading, deep fills in the swales, for example, 75 feet deep on Lot 38 and some reshaping of the natural ridge side slopes in order to construct 2.5:1 horizontal to vertical slopes. The slide areas apparently are outside the limits of old fill as indicated by Figure 2. This figure also shows the limit of the slide as it existed in 1982.

No corrective action was taken in 1982 and during the winter of 1982-1983, additional movement occurred as indicated in Figure 3. This latter movement resulted in extensive damage to the house and other improvements on Lot 27, adjacent to the Younger residence, as well as to the retaining walls of the subject site.

Slope movements have occurred elsewhere within this subdivision. A 1967 report by GJJ describes the extensive erosion and several earth slides that had resulted from winter rains. The report describes erosion of fill slopes and the toes of fills and "small surficial slides (less than 1 or 2 feet deep)...on the slope behind Lots 26-29,...". It was recommended that these slides "should be repaired by cutting back the slope to remove the slide debris, being careful that the slopes are not steepened". The report also mentioned the occurrence of the large earth slides on the cut slope encompassing Lots 14, 15, 39 and 40. The slide scarp was 5 to 6 feet high, with the toe of the slide extending nearly to the toe of the cut slope. Subexcavation and recompaction to create a drained, buttressed fill was recommended.

A report of a mudflow on Lot 37 was prepared by the Firm of Daniel J. Rhoades & Associates on March 13, 1975. The mudflow had occurred within a repaired slide zone and resulted from improper slide stabilization.

REGIONAL GEOLOGY

Setting

The project site is situated in the central part of the Coast Range Province which extends from the Oregon border to the Transverse Ranges in Southern California. In terms of geologic time the landscape is relatively young. The Coast Range Province is characterized by a series of rugged, subparallel, northwest-trending mountain ranges and intervening valleys. This regional landscape is reflected in the study area by the Santa Cruz Mountains which trend northwest and parallel to the San Andreas fault zone.

Tectonics

Mountain building, erosion and fluctuations of sea level have created the present landscape. The geologic structure of the area is complex with the northwest-trending San Andreas fault perhaps being the most prominent feature. This particular fault actually marks the boundary or separation between two plates or pieces of the earth's crust. Presently the two sections of the crust are moving past each other exhibiting horizontal and often vertical movement. During the course of this slow movement, the plates may become stationary or "locked" along a portion of their border; when the two sides finally break apart, a series of shock waves are released resulting in an earthquake. The San Andreas fault is bounded by many small faults or partial breaks in the crust; some of these are considered active and capable of generating severe earthquakes.

Geomorphology

The same type of earth energy causing movement of separate blocks of the earth's crust may cause slow vertical movement of the crust conducive to formation of mountains and valleys and folding or tilting of rock and sediments. The more recent sediments covering the present-day valley floors have been derived from erosion of the surrounding mountains. This erosion results from weathering of rock material, chiefly by water, and transportation of the eroded material downslope via streams. Under certain conditions, large portions of bedrock and soil may move down hillsides as landslides or mudflows.

These forces of movement and erosion are natural geologic phenomena. They become hazardous when development by man interacts with the natural processes. When building in a geologically active region such as the San Francisco Bay area, careful planning must be done to avoid potentially hazardous geotechnical conditions.

Structure

The geologic structure of the area is complex and known only on a regional basis. The structure has been molded by orogenic events and is characterized by extensive folding, fracturing and faulting of variable intensity. Regionally, the folds and faults trend northwesterly. This trend has been responsible for the erosional development of the region's pronounced northwest-trending ridge-valley system.

The oldest bedrock formations of the study area (Jurassic-Cretaceous) have been subjected to repeated episodes of deformation and intensely and complexly folded, faulted and deformed. In comparison to the older bedrock formations, the youngest formations (Late Quaternary) have been only mildly flexed.

The effects of ground shaking resulting from a major seismic event, such as was experienced along the San Andreas fault in the San Francisco Bay area during the earthquake of 1906, can be estimated from historical records of the subsequent damage. Earthquake intensity maps of the area have been developed from recorded damage reports of the 1906 earthquake. The effects of such earthquakes can be categorized by employing the Modified Mercalli Intensity Scale. This scale is a subjective measure of ground shaking upon the earth's ground surface, buildings, structures, and upon man's general well-being. It is not a measure of actual ground acceleration. These estimates should be used only as a general guide to help predict the possible effects of future ground shaking at a given site. If an earthquake similar to the 1906 event should be experienced along the San Andreas fault in the San Francisco Bay area, structures on the property should be expected to experience relatively strong ground shaking intensities sometime during their economic life.

Based on work by Schnable and Seed (1973), and modified by Ploessel and Slosson (1974), quantitative estimates of dynamic characteristics of a magnitude 8+ earthquake on the San Andreas fault can be approximated for the subject property. These characteristics are applied to bedrock motion, and are expressed in terms of acceleration of gravity (g). Short-term peak ground accelerations in excess of 0.6g and repeated ground accelerations for long durations of tens of seconds in excess of 0.3g are believed to be reasonable estimates for an earthquake on the San Andreas fault similar in magnitude to the 1906 event. These figures are empirically derived from typical bedrock accelerations and do not necessarily reflect ground surface accelerations modified by topography and soil-structure interaction. It should also be stressed that these values are not design parameters, but rather are intended to give a quantitative idea of reasonable expected ground responses.

SITE GEOLOGY

According to published mapping by Dibblee (1966) and Rodgers and Williams (1974), the site area is underlain by non-marine sedimentary rocks of the Santa Clara Formation. A geologic contact with an unnamed siltstone and sandstone unit lies immediately west of the slide area. A regional geologic map is presented as Figure 4. The rock types are described by Dibblee (1966) as follows:

Santa Clara Formation

Terrestrial sedimentary rocks, weakly consolidated. Rests with profound unconformity on Franciscan formation and Tertiary formations. Mostly gray to olive-brown conglomerate or gravel composed of subrounded pebbles and cobbles derived from Franciscan formation to southwest, in matrix of soft gritty sandstone or siltstone. Interbedded in lesser amounts are olive-gray to buff, soft, fine- to coarse-grained sandstone, greenish-gray to brown, gritty, soft siltstone and clay. In a few places, lignite coal occurs as layers less than 1 inch thick in siltstone.

Unnamed Sandstone

Marine, light gray, massive to poorly bedded, soft, mostly fine-grained sandstone, in places contains concretions.

The structure of the area is indicated by Dibblee's (1966) cross-section, presented as Figure 5. The section indicates that folding and faulting has occurred within the area, resulting in synclinal and anticlinal folds. Bedding in the site area is mapped as striking northwest and dipping 35° to the northeast. The 1982 Cleary report indicated that bedding exposures within their exploratory trenches were indistinct, with one exposure having an apparent dip of about "30° to the north". The ridge trends northeast-southwest, thus a northeasterly bedding dip would not indicate a dip-slope condition.

The San Andreas fault zone is located approximately 3.5 miles southwest of the site area. It is the dominant active fault in the San Francisco

Bay area, and was the source of large earthquakes in 1838 and 1906, that were accompanied by surface fault rupture. A dashed trace of the Monte Vista-Shannon fault is mapped as approximately 500 feet to the northeast of the site. The Monte Vista fault is known to have disrupted Quaternary age terrace deposits, but has not been included within the Special Studies Zones maps for potentially active faults. Subsurface investigations in the Monte Vista foothill area, about 1.5 miles northwest of this site, are reported to have proven recent activity upon the Monte Vista fault.

According to a publication by the U. S. Geological Survey (Borcherdt et al, 1975), the area under study is classified in a "C" intensity zone for large earthquakes derived from the San Andreas or Hayward faults. This value is equivalent to a value of 8 on the Rossi-Forel scale, and a value of VIII on the Modified Mercalli scale.

The National Science Foundation and National Bureau of Standards commissioned the Applied Technology Council to prepare the Tentative Provisions for the Development of Seismic Regulations for Buildings, which was published in June of 1978. The maximum horizontal acceleration in rock was established at 0.6g for the site area, which would provide a design value of 0.4g, using the Ploessel and Slosson (1974) recommended two-thirds of peak acceleration for the sustained design value.

Greensfelder, in the "Maximum Credible Rock Acceleration from Earthquakes in California" (1974), used the work of Schnabel and Seed (1973), and established a set of curves relating peak acceleration in rock, distress from fault rupture, and magnitude to plot peak rock accelerations anticipated for the major faults in California. Accelerations higher than 0.5g, the highest value that was contoured on that map, are expected to occur. However, observations of ground accelerations greater than 0.5g were too rare to be certain. Therefore, accelerations greater than 0.5g were not plotted on Greensfelder's map.

Such maximum accelerations can be determined from curves (such as those by Schnabel and Seed, 1973), which relate distance from the fault causing the earthquake, and the magnitude of the earthquake, to anticipated maximum ground acceleration in rock formations during earthquakes. Such curves, which were developed after the 1971 San Fernando earthquake, indicate that peak accelerations in excess of 0.5g can occur within a few miles of a fault or epicenter for a moderate or strong earthquake. the curves used two bands, with the upper band recorded for harder materials and the lower band for softer ones, with a maximum acceleration range of approximately 0.6g to 0.33g at a 7-mile distance from the epicentral region. The authors discussed the marked variations in rock motions as follows:

"It may be seen that the difference between upper and lower bound values of maximum acceleration at any given distance from the epicentral region is a hundred percent or more and that in fact, differences of this same magnitude may occur between the maximum accelerations in directions at right angles to each other at any one station. From the rock types at the recording stations noted on the figure it may also be seen that both upper and lower bound values were recorded on harder and softer rock types and on both the upthrown and downthrown sides of the fault. Such variations make the prediction of maximum rock accelerations for any given site and for any given earthquake extremely difficult, and similar variations can be expected to occur in frequency characteristics. Thus it would seem that at the present time probabilistic approaches provide the most rational method of assessing the nature of earthquake motions likely to be developed in rock at any given site. Past records can provide a guide to the general characteristics of the motions and the probability of their occurrence, but a suite of motions having the desired characteristics are likely to be necessary to anticipate the full range of shaking effects which might develop for any given site. Such a suite of motions may be obtained either by appropriate modification of existing records or by generation of artificial records using random process theory. Both procedures have been used successfully for analytical purposes.

While the above effects are controlled mainly by the regional geology of an area, the intensity of rock motions may also be influenced by local characteristics. For example Okamoto and Mixukoshi (1967) have reported intensities of earthquake motions twice as high on cracked and weathered rock as on sound bedrock.

On the other hand, significant variations in the characteristics of rock formations underlying soil deposits will often have negligible effects on the characteristics of ground surface motions.

In some cases the presence of a softer layer of rock may influence the frequency content of the ground surface motions without having any significant effect on the maximum acceleration value. Computations of the response of a 300 foot layer of sand underlain by rock formations consisting of (1) hard rock; (2) 500 ft. shale and then hard rock; and (3) 1000 ft. shale and then hard rock to an earthquake excitation causing a maximum acceleration of 0.1g in an outcrop of the hard rock formation lead to the following values of maximum acceleration at the surface of the sand:

Sand with hard rock base	0.777g
Sand with 500' shale and hard rock base	0.084g
Sand with 1000' shale and hard rock base	0.078g

Again the differences in maximum accelerations are less than +5% of the mean, and the response spectra for the cases with hard rock base and 1000 ft shale were quite similar. However the response spectrum for the case with 500 ft shale showed marked differences in some frequency ranges which could significantly affect the performance of structures (Lysmer et al, 1971).

Thus while variations in local rock conditions may often be of minor significance in computations of ground response, cases will occur where their effects will need to be considered for design purposes."

The foregoing research indicates that differences in opinion exist among the major contributors in the area of rock acceleration derived from a major seismic event.

The moderately indurated nature of the Santa Clara formation and the variability in lithologic character, makes for variances in general slope stability. Clay portions of the formation are moderately to highly plastic and expansive, as indicated by Cleary's Atterberg Limits test showing a liquid limit of 47 and plasticity index of 28. The sample tested also had a free swell of 50%, also indicative of high expansive properties. A regional slope stability report of the San Francisco Bay region by Nilsen and others (1979) indicated that the general site area is "generally stable to marginally stable". However, areas in the vicinity of the site are labeled as "unstable", in "areas that are underlain by or immediately adjacent to landslide deposits. Much of this study was based upon analysis of aerial photographs, and thus large swale areas may be labeled as landslides.

A statistical study of landsliding and different geologic units on different slopes, determined that the Santa Clara formation is least susceptible to landsliding on slopes between 0 and 15 percent; moderately susceptible on slopes between 15 and 30 percent; and has a moderately high susceptibility to landsliding upon slopes exceeding 30 percent (Brabb et al, 1972).

SITE-INVESTIGATION

Due to the steep terrain, a porta-sampler was used to obtain a 1-inch diameter continuous sample of the subsurface soils. The sampling energy source was a gas-operated impact hammer. The number of blows to penetrate 1 foot of soil was recorded with time. The drilling company stated that a penetration rate of 8 seconds per foot was roughly equal to a Standard Penetration blow count of 10 blows. In any event, the recorded blow counts on the attached boring logs are useful in determining relative consistency and strength. The Logs of Test Borings are presented as Figures 6 through 12.

The obtained samples were taken to the laboratory for detailed scrutiny, soil classification and laboratory testing. Careful "dissection" of the soil samples allowed greater detail in determining weak layers and description of soil structure. The test borings were oriented such that a continuous profile could be prepared, as shown on Figure 13.

No free groundwater was encountered in the test borings, however seepage is expected and isolated areas of high moisture content also indicate the presence of seepage. The exploratory trenches performed by Cleary Consultants also did not encounter a free groundwater table, but seepage was indicated in one excavation.

DISCUSSION OF FIELD INVESTIGATIONS

Test borings through the length of the slide encountered mottled orange-brown, reddish and olive-brown clayey sand, sandy clay and gravelly clay. This is in agreement with the materials encountered in Cleary's test pits. The blow counts and moisture contents indicated on the attached Logs of Test Borings, Figures 6 through 12, generally indicate the contact between the slide debris and the firm underlying strata. The clayey sand and gravelly clay layers allow the passage of seepage, which then saturates and reduces the shear strength of sandy clay layers. Slip surfaces noted in the Cleary report were only 1/8-inch thick within a sandy clay layer.

The cross-section shown as Figure 13 indicates that the depth of the slide varies from about 12 feet below the existing ground surface at midslope to being exposed in the headscarp. The slide debris continued downslope and turned to the northeast, into Lot 27, causing extensive damage to the house and rear-yard improvements.

LABORATORY TESTING

Appropriate laboratory testing on undisturbed soil core samples was performed to determine the in-situ moisture and density characteristics, unconfined compressive strength and expansion characteristics.

One of the most important engineering properties of soil is its shear strength, or its ability to resist shear forces along internal planes within the hillside and soil mass. It is the property which enables a soil mass to maintain equilibrium on a sloping surface such as a natural or manmade embankment. The actual and the uniformity of the shear strength of a soil mass is one of the more difficult problems in soil mechanics testing because shear strength is not an intrinsic property of a given hillside soil, and will vary over a considerable range with varying conditions such as density, moisture content and degree of saturation for each localized soil type. The original soil investigation by GJJ included direct shear test results. The strength characteristics of both the in-place and remolded near-surface were determined by placing specimens in contact with water at least 24 hours before testing, and then sheared under normal loads ranging from 1000 to 4000 p.s.f. Remolded samples were prepared at approximately 90 percent relative compaction. The test results are presented as Figure 14. The test results from this recent study are presented with the Logs of Test Borings, Figures 6 through 12.

GEOTECHNICAL CONCLUSIONS

The materials encountered in the test borings, observed during surface reconnaissance and studied in published data indicate the property is underlain by a moderately indurated bedrock type of variable lithology and stability. Test borings encountered a bedrock which varies from dense and brittle to deeply weathered and expansive clay soils.

The results of this study, and the results of past investigations performed by other geotechnical firms on and near the property indicate that this site and portions along the remainder of this hillside have experienced past landslide activity. The bedrock type found at this site has been involved in slope failures in the past due to its rapid breakdown into a weakened condition comparable to a clay matrix during saturation. These clays frequently contain slickensides or fissures which are caused by uneven volume changes during weathering. Terzaghi (1936) points out that it is by no means uncommon that the shear strength of a stiff fissured clay may be decreased by the infiltration of water into the cracks. Most slides in stiff fissured clays are probably affected by progressive failure. Failure generally takes place at an average shear stress between the peak and the residual drained shear strength.

In light of this study, the following factors would seem appliable to the landslide movement and mechanism under study:

1. Grading of the site resulted in constructing a cut slope in material that has seepage zones within gravelly clays and clayey sand layers and weak, expansive clay soils.

2. Several past slope movements in the tract area have occurred on both cut and fill slopes. The past failures were attributed to insufficient drainage, and improper grading procedures on repaired slopes.

3. The bedrock at the site is weakly to moderately indurated and characteristically breaks down into lithologies resembling colluvial soils. This rock type has seepage zones within the more granular layers which saturates expansive clay beds. Because of this characteristic breakdown, this bedrock is recognized as being susceptible to slope instabilities. This unit in a dry condition has fair slope stability, but shear strength of this material decreases rapidly in a saturated condition. The extensive and deep fissures that are a result of the expansive clays, extend to the surface and become filled with both subsurface and surface water and thus, progressively weathers, creeps downslope and eventually fails.

In view of the fact that this downslope movement of the landslide mass involves in-place weathered and sheared bedrock materials, any engineering design employed to promote stability must take this into account.

In light of the poor stability afforded by the on-site soils, the recommended remedial design should not utilize these soils as backfill materials without altering their physical characteristics, either by blending-in an import admixture or the sole use of import. Therefore, two alternate repair solutions are provided: either a buttressed fill repair or a soldier beam retaining walls with granular backfill. The location of the soldier beam retaining walls would be in the general area of the main toe at the rear-yard of Lot 28 and at mid-slope, these walls can be extended over to Lot 27 in order to stabilize that portion of the slide.

The location of the walls are approximately shown on Figure 15. The basic scheme would be to install a wall along the toe of the unstable area and at mid-slope. The unstable area would then be excavated out to firm material and then the area between the walls backfilled with a select import of 12-inch riprap. The backfill behind the upper wall would be of blended import. Tiebacks and soldier beam piles would be drilled with the soldier beams placed on approximately 8-foot centers. This method will derive its restraint from the tiebacks and the fixed end condition of the soldier beams placed in the bedrock at depth, beyond the influence of near-surface weathering. Specific design parameters and procedures for the tiebacks, and pile installation, wall construction, earthwork specifications and both surface and subsurface drainage control are included in the Soil Engineering Considerations of this report.

The installation of an engineered, buttress fill alternate includes the removal of the slide debris down to competent material, installation of a deep keyway at the base of the new fill slope including gravel blanket and subdrains, followed by engineered fill placed at a slope inclination considered stable for existing conditions. The vibratory nature of the construction equipment, in addition to the necessity of removing the slide debris which currently serves as a restraining force for equilibrium purposes, causes a potential danger to the hillside above. Therefore, we are of the opinion that the landslide repair will necessitate a consideration of the alternates, with economics and the necessity for rapid implementation prior to the foreseeable winter rain period being the major consideration in the ultimate decision of the successful alternate. We envision combinations of internal drainage features, engineered fill and structural walls as being the successful combination to replace stability to the hillside. These recommendations are provided in the detailed recommendations in the Soil Engineering Considerations section that follows.

SOIL ENGINEERING CONSIDERATIONS

The geologic interpretation of the earth movement concluded that the failure mode consisted of internal saturation from above reducing the strength of the near-surface weathered in-place bedrock to a safety factor less than unity. The sheared and weathered nature of the underlying Santa Clara formation, in conjunction with the groundwater conditions, weakened the supporting materials to a point where the shear strength was exceeded, resulting in a failure of the hillside. The final repair method must therefore address the cost of correcting the defect and the utilization of the stable bedrock materials located at depth.

Of primary importance in the reconstruction of the hillside will be the careful attention to good workmanship standards to ensure that all measures are taken to construct the repaired slide in accordance with the standards set forth in this report. The final contractor must be experienced in hillside repairs similar to the proposed solution. The slide repair must be initiated prior to the forthcoming seasonal rain period, as the rapid placement of the tiebacks and soldier beam piles and/or buttress fill are essential to prevent further instability above during the repair work. Full-time inspection by geotechnical personnel from this office is mandatory to ensure that the design conditions have been met.

The following is a description of the construction methods or procedures that may be utilized with each alternate. The construction difficulties associated with each alternate is affected by the confined working areas and the deep excavation required for the buttress fill alternate. It would be most difficult to properly key and develop a uniform fill buttress behind only the Younger residence in the event the adjacent property is not stabilized at the same time. The structural wall alternate addresses this issue and may be utilized to accommodate building to the property line and thus, stabilize the area above and not encroach upon the adjacent property.

Alternate 1: Buttress Fill

After clearing the surface debris, the placement of gallery drains at 30 feet on-center should commence, with the drains extending through the slide debris 4 feet into the parent bedrock material below. The drains should be backfilled with Class II filter rock and extend to the proposed exit drain line at the main keyway location at the rear property line. Upon completion of the gallery drains to drain the slide debris, leaving portions of the slide debris in place, the main keyway should be excavated in short stable reaches (30 to 40 feet) for a minimum depth of 10 feet into the bedrock unit comparable to the cross-section shown on Figure 16.

As the fill is placed in the keyway and commencing the fill prism upslope, the fill shall be placed at a minimum compaction of 95% and shall consist of imported material blended with on-site materials to develop a minimum shear strength of 750 p.s.f. with an angle of internal friction of 20° or the use entirely of import with wastage of excavated spoils. As the fill is brought upslope, the slide debris will be removed prior to placement of the fill between the gallery drains. All fill must be placed upon level keyways with geofabric placed over the gallery drains prior to progressive in-filling of the hillside. Caution must be maintained to prevent undermining the slope. This risk must be borne by the Contractor in choosing his equipment and procedures to regrade the hillside.

Alternate 2: Soldier Beam Retaining Wall

This alternate consists of drilling caissons and installing soldier beam retaining walls utilizing tiebacks along the rear-yard areas of Lots 27 and 28 and at a location approximately mid-slope. The intent of this alternate is to be able to utilize 12-inch angular riprap for the slope between the two walls, at a finish slope of 1-1/2:1 horizontal to vertical or less. The slope behind the upper wall would utilize a blended backfill for a finish slope of 2.5:1 or flatter. The schematic diagram illustrating this alternate is presented as Figure 17. A possible alternate not shown could incorporate the lower soldier beam wall, with the angular riprap placed to mid-slope, above which would be the installation of a keyed-in buttress fill for the remainder of the slope. These alternates are intended to give guidelines to an experienced contractor who is familiar with his material sources and equipment.

With the foregoing considerations serving as an outline to the development of the property, the following recommendations are provided:

A. SITE GRADING AND LANDSLIDE REPAIR

1. General - Prior to the construction of whichever alternate is selected, the general area where the improvements are to be located must be cleared of all surface slide debris down to stable bedrock materials. This will necessitate special caution in operating equipment where excessive vibratory motion might endanger the stability of the hillside if seepage conditions are encountered. We envision sequential events where portions of the slide debris is removed and backfilled prior to removal of the next stage to finally include the complete removal of the unsuitable material for the full width of the fill placement.

2. Internal Drains - Gallery drains may be placed proceeding this clearing operation in order to predrain the area and thus, permit cleaning of the debris material in sections between the gallery

drains. The gallery drains are to consist of Class II permeable filter material placed within excavated trenches, penetrating a minimum depth of 4 feet into underlying bedrock materials. These drains should slope downward toward the main keyway and thus, connect into the main drainage system for the entire slide repair.

The main keyway is to be constructed using a 20-foot wide base keyway, extending at the shallowest location a minimum depth of 10 feet into bedrock materials, having a strength and acceptability as determined by the Soil Engineer in the field. This keyway is to slope inward at an inclination of 2% where Class II filter rock will be placed at the uphill intercept of the keyway where an 8-inch perforated pipe will be located to collect all internal seepage water. The configuration of the main keyway and subdrain system shall permit gravity flow to a discharge point downhill from the main slide that will be subsequently connected into a line discharging to Lindy Drive. After the keyway drain has been installed, the initial keyway should be backfilled and compacted to a minimum of 95% of ASTM D1557-78 to the finished grade surface elevation. As the slide repair progresses uphill from the main keyway, intermediate keyways shall be constructed, all sloping inward to the hill at a 2% gradient for a minimum width of 8 feet at vertical heights of approximately 4 feet to adequately support the main buttress fill. Where localized soft spots are encountered, deeper excavations will be required. Within the transition zone from the main keyway uphill, a center drain will be provided, enclosed in Class II filter material draining each bench into the main drain below, exiting into the discharge line as provided during the initial construction of the keyway.

Upon completion of each drainage installation and the removal of the slide debris as the fill progresses, all material shall be compacted to a minimum relative compaction of 95%. Where localized seepage areas are encountered or spring activity is anticipated, gravel blankets and perforated pipe will be installed, all connecting into the main drain. Caution must be utilized in the transition zones above the main keyway and on each lateral extension of the intermediate keys to ensure that the keys are placed deep enough and that all unstable material has been removed down to firm non-yielding materials.

3. The main fill shall extend across the entire width of the failed area and shall include repairing all unstable areas to the rear of both the Younger and adjoining residence. This method is not a viable solution in the event the adjoining residence is not

included into the overall restoration of the hillside. The finished slope should be overfilled with final dressing consisting of a smooth and uniform cut, with a slope inclination of 2.5 horizontal to 1 vertical into well-compacted materials. Final planting is required as specified by a certified California nurseryman or landscape architect.

4. General grading specifications are included in the attached appendix, which set forth guidelines for the fill placement not specifically enumerated in the text of this report. No fill shall be placed during inclement weather and the main buttress fills must be placed prior to the seasonal rain period.

5. All grading operations must be under the observation of an Engineering Geologist or Soil Engineer, in addition to the normal compaction testing procedures conducted by a field Engineering Technician.

B. RIPRAP EMBANKMENT REPAIR

1. The riprap embankment repair will generally consist of the similar type installation as for the engineered buttress fill, with the exception that the exterior slope inclination may be at an inclination of 1-1/2 horizontal to 1 vertical or less, and the fill

material may consist of well-graded, durable quality materials generally defined as riprap ranging from 12 inches down to 4 inches in gradation. The entire installation of keyways and internal keyways will be required, with all rock materials placed upon geofabric to prevent infiltration of fines and loss of support of the hillside above. A surface soil cover shall be placed over the rock fill to permit plant growth and to provide for surface storm water runoff. Internal collector drains will be required and generally consist of the same piping system as defined under Item 1 above. All removals of slide debris, installation of gallery drains or other features will similarly be incorporated into this unit.

C. SURFACE DRAINAGE

1. Only water falling directly upon cut or fill slopes should be permitted, and all water falling upon such slopes where retaining walls are located below slopes should be channelled into a controlled system to prevent hydrostatic forces from building up behind the retaining walls. No ponding of water can be permitted, above the slope or upon the pads or the adjacent improvements, that would permit the storm water to percolate into the slope mass, causing excessive saturation of the soil embankment.

2. All structures must have gutters and downspouts discharging their effluent into a suitable drainage control system to not only direct the water away from the foundation areas, but also to prevent excessive saturation of the adjacent slopes. Where suitable discharge points can be designed, tightlines connecting to such facilities would satisfy this requirement. All exposed slopes must be planted as soon as practical after completion of construction to ensure sufficient growth prior to the next rain period, and thus prevent erosion problems from developing. Hemp impregnated blankets have been found excellent to provide early growth on erosion-prone slopes. A landscape architect should be consulted concerning erosion protection procedures.

D. RETAINING WALL DESIGN

1. Retaining walls may consist of soldier beam with tiebacks. The commencement of the passive pressure shall begin at a depth of 6 feet into the bedrock.

For a conventional drained wall using granular backfill, an active pressure of 45 p.c.f. should be used. A 100 p.s.f. surcharge should be added to the 45 p.c.f. fluid pressure to finalize the active pressure for design purposes.

2. Seismic considerations should include an additional factor of 30% of all active forces, including slope surcharge load.

3. The passive forces to resist the active condition may be taken as a minimum bearing pressure of 2000 p.s.f., commencing below the 6-foot level into bedrock, acting over 2 times the diameter of the caisson. The tiebacks may be designed for a frictional value of 750 p.s.f. adhesion, penetrating a minimum depth of 10 feet into bedrock materials to initiate the start of the adhesion forces. Regardless of the length calculations, all tiebacks should extend a minimum depth of 50 feet into the hillside and must be pressure grouted under the observations of the Soil Engineer. All tiebacks are to be given a proof load equal to 1.3 times their designed working load as observed and verified by the Soil Engineer.

4. The soldier beam retaining members or caissons must be inspected by the Soil Engineer during drilling to ensure that the recommended values are obtained in the field and that refusal or other conditions do not dictate shallower piers. In general, where the minimum depth places the caisson upon the bedrock, an additional depth factor will be determined in the field to ensure adequate keying between the two supporting materials.

5. The above active pressures are for a drained wall which will require use of granular backfill and drainline to ensure rapid drainage of materials behind the walls. Weep holes are not recommended, as the moisture discharge will tend to weaken the fill embankment through prolonged saturation. A drainage swale at the base of the wall is necessary, and preferably an impermeable drainage ditch adjacent to the wall and slope intercept, to carry off the surface runoff from above to properly placed inlet structures or collection boxes for disposal.

6. The sequence of constructing the retaining walls is critical as discussed under "Soil Engineering Considerations", with the final construction method to be reviewed and approved by the Soil Engineer to ensure that the wall and adjacent slopes are properly protected.

Erosion protection as described above should be incorporated as soon as practical to prevent surface damage to the exposed embnkments.

E. SUPPLEMENTAL REPORT

1. A final report will be submitted upon completion of the grading operations and the installation of the surface and subsurface

drainage facilities locating all facilities on an accurate "as-built plan".

LIMITATIONS AND UNIFORMITY OF CONDITIONS

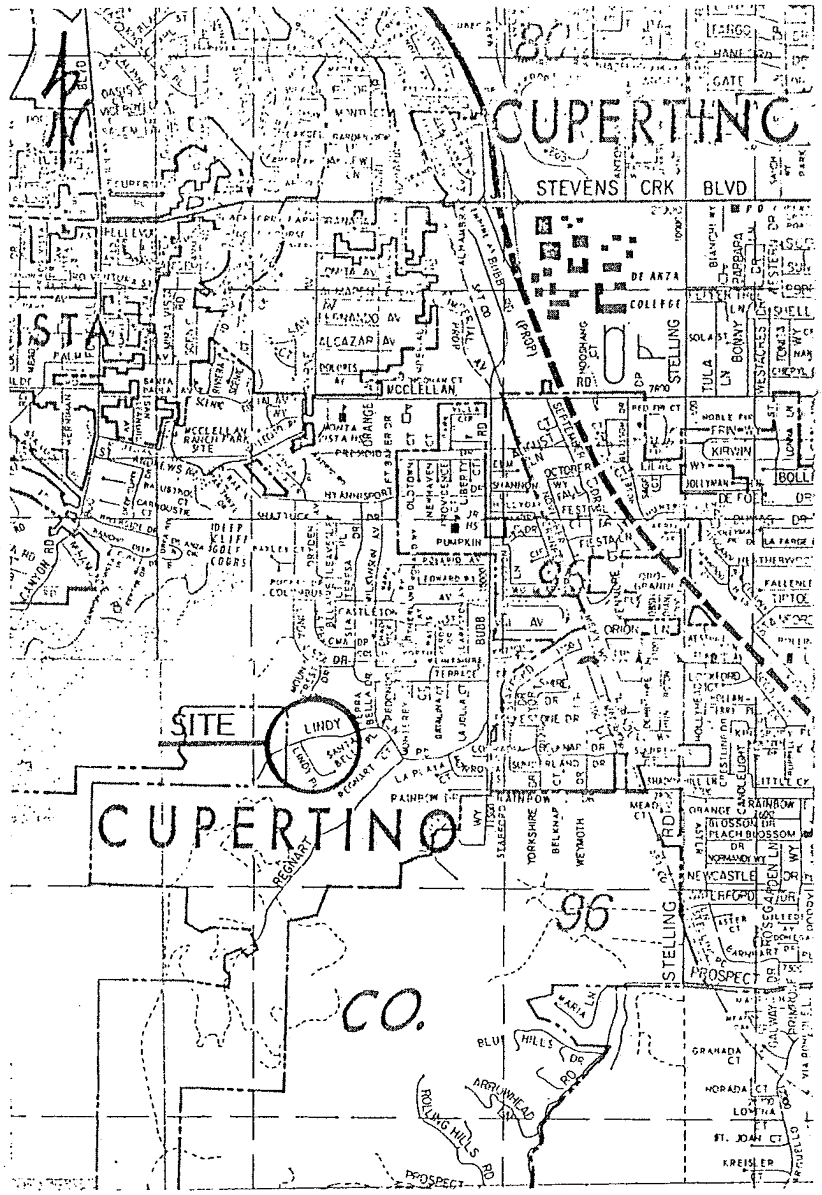
Physical changes to a property, from the condition at which it existed during the time our subsurface exploration was accomplished, can subsequently be brought about by natural or manmade causes. Additionally, the standards of work which are acceptable to approving agencies may be raised during the passage of time and what is acceptable to the approving agency at this time may not be in the future. For these reasons, the recommendations contained in this report are valid for a period of one year, after which time they must be reviewed by our firm to determine whether or not they are still applicable.

Our services consist of professional opinions and recommendations made in accordance with generally accepted soil mechanics, foundation engineering and engineering geology principles and practices. This warranty is in lieu of all other warranties either expressed or implied. The recommendations submitted in this report are based upon information obtained from our review of published geotechnical data, site reconnaissance, subsurface exploration, laboratory testing and appropriate analyses. Unanticipated soil conditions are frequently encountered and cannot be fully determined by excavating test pits or drilling and sampling test borings, and may require that additional expenditures be made during the construction phase of the project to obtain a properly built project. It is recommended that you establish some contingency fund to accommodate these extra costs if they become necessary.

This report has been prepared in order to aid in the evaluation of this project. In the event any changes in the proposed development concept or location of the facilities are planned, our conclusions and recommendations should not be considered valid unless the changes are reviewed and our conclusions modified or approved in writing by us. It is your responsibility to ensure that our recommendations are made available to your Project Architect, Project Engineer and Contractor.

FIELD EXPLORATIONS
LABORATORY TESTING

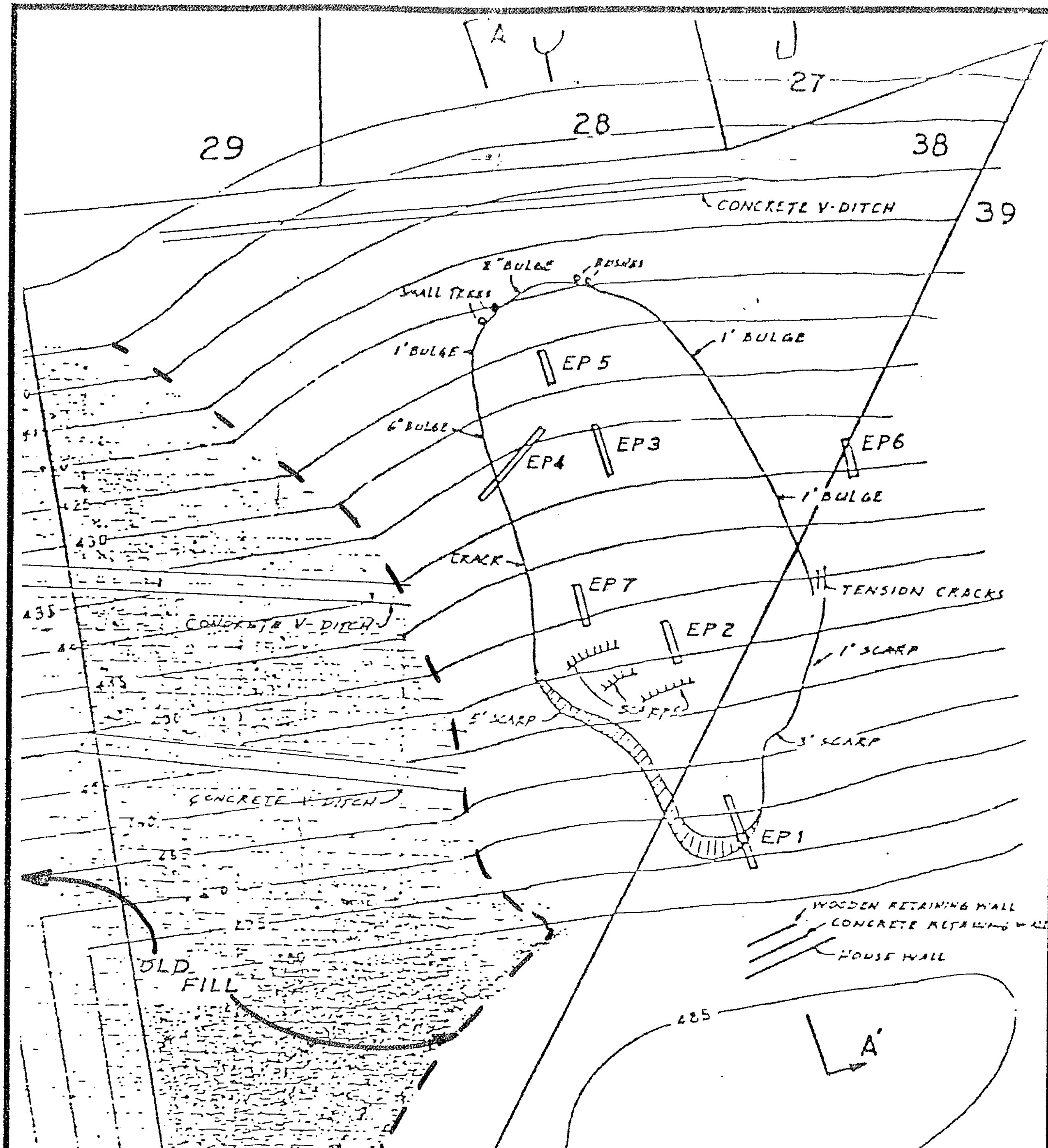
FIELD EXPLORATIONS
LABORATORY TESTING



*Source: Thomas Bros. Map, Santa Clara County

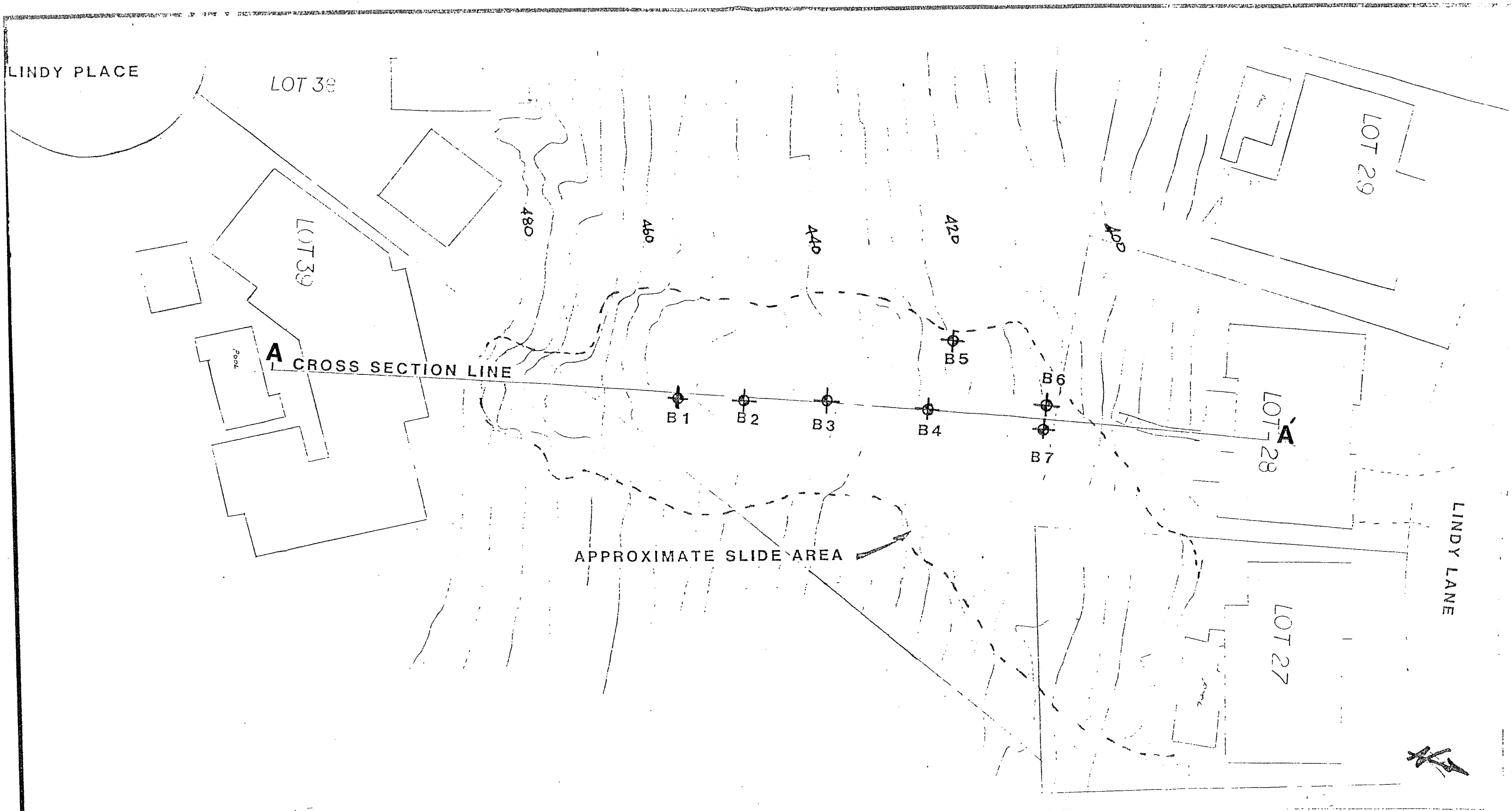
FIGURE I	**PURCELL, RHOADES & ASSOCIATES** **Foundation Engineering • Soil Engineering • Geology**
	SITE LOCATION MAP*

DATE 9/15/83	DRAWN BY IDA	CHECKED BY DJR
SCALE 1"=2200'	W.O. 3179	



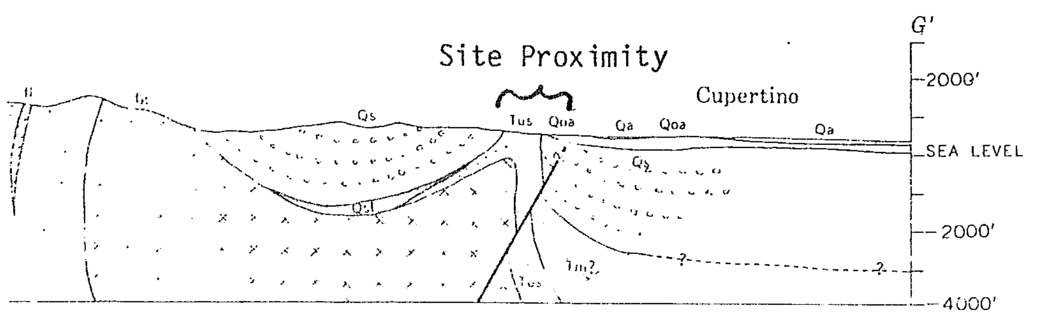
*Base: George S. Nolte Plans and Profiles for Tract 3354, 1965. Reproduced from Report by Cleary Consultants October 1982. Test Pit Logs Presented in Figure 18.

FIGURE 2	**PURCELL, RHOADES & ASSOCIATES** Foundation Engineering • Soil Engineering • Geology ORIGINAL CONTOURS AND LOCATION OF 1981-1982 SLIDE*		
DATE 9/15/83	DRAWN BY IDA	CHECKED BY DJR	
SCALE 1" = 40'	W.O. 3179		



*Portion of Base Map Prepared by Frahm-Edler-Cannis, July 11, 1983, Job No. 3795

FIGURE 3	PURCELL, RHOADES & ASSOCIATES Foundation Engineering • Soil Engineering • Geology SITE PLAN*	
DATE 9/15/83	DRAWN BY IDA	CHECKED BY DJR
SCALE 1"=32'±	W.O. 3179	



*Source: Dibblee, 1966

FIGURE 5	PURCELL, RHOADES & ASSOCIATES Foundation Engineering • Soil Engineering • Geology PORTION OF CROSS-SECTION G-G'		
DATE 9/15/83	DRAWN BY IDA	CHECKED BY DJR	
SCALE 1"=1 mile	W.O. 3179		

DRILL RIG Porta-Sampler	BORING ELEVATION Ext. Grade	LOGGED BY STD	PROJECT NO. 3179	BORING NO.
DEPTH TO GROUNDWATER None	BORING DIAMETER 1"	DATE DRILLED 8/4/83	SHEET 1 OF 1	1

EXPLORATORY BORING LOG

SOIL/ROCK DESCRIPTION—CLASSIFICATION AND REMARKS	CONSISTENCY	GROUP SYMBOL (U.S.C.S.)	WATER LEVEL / GRAPHIC LOG	DEPTH IN FEET	SAMPLE	BLOW COUNTS / FOOT	% MOISTURE CONTENT	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (PSF)	PLASTICITY INDEX (PI)
Slide Debris - Mottled orange-brown and light gray-brown Silty and Sandy CLAY with rock fragments	loose			1						
				2						
				3						
				4						
				5		4				
				6		9				
Slide Plane				7						
Santa Clara Formation - Moderately indurated, orange-brown and gray-brown Sandy CLAY and Gravelly CLAY, stiff to very stiff	stiff					16				
				8						
				9						
	very stiff			10		36				
				1						
				2						
				3						
Boring Terminated at 13.0 feet				4						
				15						
				6						
				7						
				8						
				9						
				20						
				1						

PURCELL, RHOADES & ASSOCIATES
FOUNDATION ENGINEERING § SOIL ENGINEERING § GEOLOGY

TITLE FIGURE 6 - LOG OF TEST BORING 1

DRILL RIG	BORING ELEVATION	LOGGED BY	PROJECT NO.	BORING NO.
Porta-Sampler	Ext.Grade	STD	3179	2
DEPTH TO GROUNDWATER	**BORING DIAMETER**	**DATE DRILLED**	**SHEET**	
None	1"	8/4/83	1 OF 1	

EXPLORATORY BORING LOG

SOIL/ROCK DESCRIPTION—CLASSIFICATION AND REMARKS

SOIL/ROCK DESCRIPTION—CLASSIFICATION AND REMARKS	CONSISTENCY	GROUP SYMBOL (U.S.C.S.)	WATER LEVEL / GRAPHIC LOG	DEPTH IN FEET	SAMPLE	BLOW COUNTS / FOOT	% MOISTURE CONTENT	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (PSF)	PLASTICITY INDEX (PI)
Slide-Debris - Mottled orange-brown Silty and Sandy CLAY with rock fragments and gravels	loose			1						
				2						
				3						
				4						
				5						
				6						
	loose			7		2	21.0	94.1	----	
				8						
				9						
				10						
				1		11	21.3	101.0	----	
				2		12	23.9	-----	----	
Wet Slide Plane	no re-covery			3		13	54.8	-----	----	
Santa Clara Formation - Mottled orange and gray-brown Silty CLAY, Clayey, Sandy and Gravelly CLAYS	firm			4		14	25.1	111.7	----	
				15						
				6						
	very stiff			7		75	20.4	109.4	----	
				8						
Boring Terminated at 18.0 feet				9						
				20						
				1						

PURCELL, RHOADES & ASSOCIATES
FOUNDATION ENGINEERING § SOIL ENGINEERING § GEOLOGY

TITLE
FIGURE 7 - LOG OF TEST BORING 2

DRILL RIG	BORING ELEVATION	LOGGED BY	PROJECT NO.	BORING NO.
Porta-Sampler	Ext.Grade	STD	3179	3
DEPTH TO GROUNDWATER	**BORING DIAMETER**	**DATE DRILLED**	**SHEET**	
None	1"	8/4/83	1 OF 1	

EXPLORATORY BORING LOG

SOIL/ROCK DESCRIPTION—CLASSIFICATION AND REMARKS

SOIL/ROCK DESCRIPTION—CLASSIFICATION AND REMARKS	CONSISTENCY	GROUP SYMBOL (U.S.C.S.)	WATER LEVEL / GRAPHIC LOG	DEPTH IN FEET	SAMPLE	BLOW COUNTS / FOOT	% MOISTURE CONTENT	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (PSF)	PLASTICITY INDEX (PI)
Slide-Debris - Mottled orange and medium brown, Silty and Sandy CLAY with rock fragments and gravels	loose			1						
	loose			5–6	I	2				
				10–11	I	10				
				11–12	I	10				
Slide Plane										
Santa-Clara-Formation - Mottled orange and reddish-brown Silty CLAY and Sandy, Gravelly CLAY	stiff			12–13	I	21				
	very stiff			15–16	I	41				
Boring Terminated at 20.0 feet				20						

PURCELL, RHOADES & ASSOCIATES
FOUNDATION ENGINEERING § SOIL ENGINEERING § GEOLOGY

TITLE: FIGURE 8 - LOG OF TEST BORING 3

DRILL RIG	Porta-Sampler	BORING ELEVATION	Ext. Grade	LOGGED BY	STD	PROJECT NO. 3179	BORING NO.
DEPTH TO GROUNDWATER	None	BORING DIAMETER	1"	DATE DRILLED	8/4/83	SHEET 1 OF 1	4

EXPLORATORY BORING LOG

SOIL/ROCK DESCRIPTION—CLASSIFICATION AND REMARKS	CONSISTENCY	GROUP SYMBOL (U.S.C.S.)	WATER LEVEL / GRAPHIC LOG	DEPTH IN FEET	SAMPLE	BLOW COUNTS / FOOT	% MOISTURE CONTENT	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (PSF)	PLASTICITY INDEX (PI)
Slide-Debris - Mottled orange and medium brown Clayey SAND and Sandy CLAY with coarse gravels	loose			1						
	loose			5–6	■	2				
Slide Plane	firm			10–11	■	11	23.8	104.3	----	
Santa Clara Formation - Mottled orange-brown and gray-brown Clayey SAND, Sandy CLAY and Gravelly CLAY	stiff			11–12	■	33	21.0	104.8	----	
	very stiff			14–15	■	40	15.2	122.1	----	
Boring Terminated at 16.0 feet				16						

PURCELL, RHOADES & ASSOCIATES

FOUNDATION ENGINEERING § SOIL ENGINEERING § GEOLOGY

TITLE

FIGURE 9 - LOG OF TEST BORING 4

DRILL RIG: Porta-Sampler	BORING ELEVATION: Ext.Grade	LOGGED BY: STD	PROJECT NO. 3179	BORING NO.
DEPTH TO GROUNDWATER: None	BORING DIAMETER: 1"	DATE DRILLED: 8/4/83	SHEET 1 OF 1	5

EXPLORATORY BORING LOG

SOIL/ROCK DESCRIPTION—CLASSIFICATION AND REMARKS

SOIL/ROCK DESCRIPTION—CLASSIFICATION AND REMARKS	CONSISTENCY	GROUP SYMBOL (U.S.C.S.)	WATER LEVEL / GRAPHIC LOG	DEPTH IN FEET	SAMPLE	BLOW COUNTS / FOOT	% MOISTURE CONTENT	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (PSF)	PLASTICITY INDEX (PI)
Slide Debris - Mottled orange and gray-brown Silty and Sandy CLAY with rock fragments and gravel	loose			1						
				2						
				3						
				4						
				5						
	loose			6–7		9	18.7	103.4	----	
				8						
Slide Plane	firm			9–10		16	17.7	112.2	----	
						19	16.1	-----	----	
Santa Clara Formation - Mottled orange-brown and gray-brown, Gravelly, Silty CLAY	stiff			1–2		29	16.3	106.0	----	
				3						
				4–15		61	13.6	119.7	----	
Gray-brown Sandy CLAY and CLAY-SAND	very stiff			6						
				7						
				8						
				9						
Boring Terminated at 19.5 feet				20						
				1						

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TITLE

FIGURE 10 - LOG OF TEST BORING 5

DRILL RIG Porta-Sampler	BORING ELEVATION Ext.Grade	LOGGED BY STD	PROJECT NO. 3179	BORING NO. 6
DEPTH TO GROUNDWATER None	BORING DIAMETER 1"	DATE DRILLED 8/4/83	SHEET 1 OF 1	

EXPLORATORY BORING LOG

SOIL/ROCK DESCRIPTION—CLASSIFICATION AND REMARKS

SOIL/ROCK DESCRIPTION—CLASSIFICATION AND REMARKS	CONSISTENCY	GROUP SYMBOL (U.S.C.S.)	WATER LEVEL / GRAPHIC LOG	DEPTH IN FEET	SAMPLE	BLOW COUNTS / FOOT	% MOISTURE CONTENT	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (PSF)	PLASTICITY INDEX (PI)
Slide Debris - Mottled orange and gray-brown Silty CLAY and Sandy CLAY with rock fragments and gravels	loose			1						
				5–6		6				
				8–9		8				
Slide Plane				9						
Santa Clara Formation - Mottled orange and gray-brown, Gravelly, Clayey SAND and Gravelly, Sandy CLAY				9–10		19	14.7	123.6	----	
				10–11		60	10.8	129.5	----	
Boring Terminated at 12.0 feet				12						

Depth scale: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 1, 2, 3, 4, 15, 6, 7, 8, 9, 20, 1

PURCELL, RHOADES & ASSOCIATES

FOUNDATION ENGINEERING § SOIL ENGINEERING § GEOLOGY

TITLE FIGURE 11 - LOG OF TEST BORING 6

DRILL RIG: Porta-Sampler	BORING ELEVATION: Ext.Grade	LOGGED BY: STD	PROJECT NO. 3179	BORING NO. 7
DEPTH TO GROUNDWATER: None	BORING DIAMETER: 1"	DATE DRILLED: 8/4/83	SHEET 1 OF 1	

EXPLORATORY BORING LOG

SOIL/ROCK DESCRIPTION—CLASSIFICATION AND REMARKS	CONSISTENCY	GROUP SYMBOL (U.S.C.S.)	WATER LEVEL / GRAPHIC LOG	DEPTH IN FEET	SAMPLE	BLOW COUNTS / FOOT	% MOISTURE CONTENT	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (PSF)	PLASTICITY INDEX (PI)
Slide-Debris - Mottled orange and gray-brown Silty and Sandy CLAY with rock fragments and gravels	loose			0						
				5		3				
				6		6	17.9	112.2	----	
				7		8	23.3	106.0	----	
Medium brown Sandy CLAY with few gravels				8		11	23.3	106.6	----	
Boring Terminated at 9.0 feet				9						

PURCELL, RHOADES & ASSOCIATES
FOUNDATION ENGINEERING & SOIL ENGINEERING & GEOLOGY

TITLE: FIGURE 12 - LOG OF TEST BORING 7

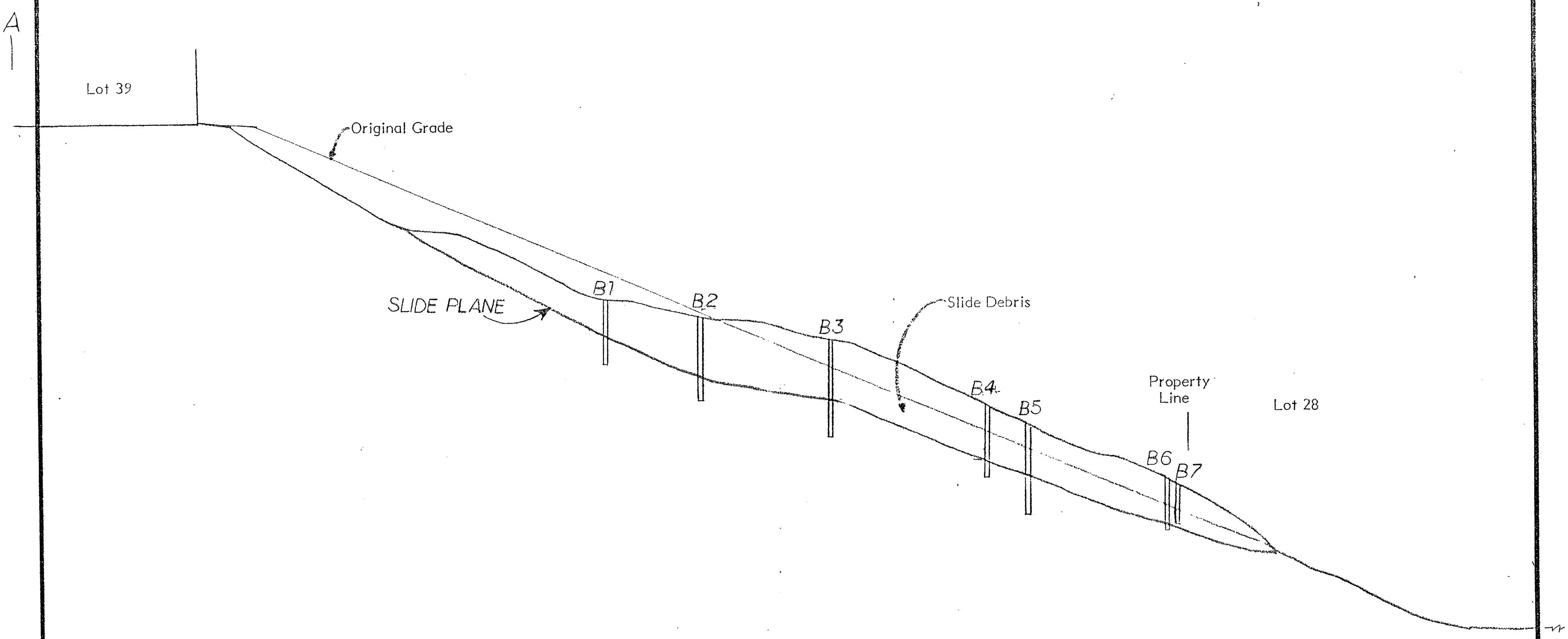


FIGURE 13	PURCELL, RHOADES & ASSOCIATES Foundation Engineering • Soil Engineering • Geology CROSS-SECTION A-A'	
DATE 9/15/83	DRAWN BY IDA	CHECKED BY DJR
SCALE 1" = 20'	W.O. 3179	

TABLE I

Summary of Plasticity Index and Hydrometer Test Results

M.I.T. Classification

Hole	Depth	Plasticity Index		Hydrometer Analysis			
		Liquid Limit	Plasticity Index	Gravel	Sand	Silt	Clay
no.	ft.	%		%	%	%	%
1	1	22.8	4	11	38	39	12

TABLE II

Summary of Moisture-Density, Swell and Direct Shear Tests

Hole no.	Depth ft.	Moisture Content %	Dry Density p.c.f.	Swell Index	Angle of Friction degrees	Unit Cohesion p.s.f.
1	0.5-1	12.2	120.3	0.5	22.5	730
1	1.5	17	97.5			
1	4.5	8	97.8			
1	9.5	7.2	95.6			
2	2.5	21.2	102.7	1.0	16	870
2	4.5	16.4	116.5			
3	1.5	20.3	106.9			
3	4.5	16.5	111.9	0.9	25	1250
4	2	15.8	98.8			
4	4.5	13.0	108.0			
4	9.5	11.5	123.0			
*5	0.5-1	11.0	129.0	1.5	23.5	860

FIGURE 14: TEST RESULTS OF GRIBALDO, JACOBS & JONES

TABLE II

Summary of Moisture-Density, Swell and Direct Shear Tests

Hole no.	Depth ft.	Moisture Content %	Dry Density p.c.f.	Swell Index	Angle of Friction degrees	Unit Cohesion p.s.f.
6	1.5	22.8	88.6			
6	3.5	23.1	97.4			
6	5.5	30.9	88.7			

Notes: Underscore indicates optimum values. See Figures No. 7 and 8 for complete curve.

* Samples remoulded to within 90% of maximum dry density at optimum moisture as determined by ASTM D1557, Method A.

FIGURE 14: TEST RESULTS OF GRIBALDO, JACOBS & JONES (Continued)

TABLE I

Summary of In-Place Moisture-Density, Swell and Direct Shear Test Results

Hole No.	Depth ft.	Dry Density p.c.f.	Moisture Content % dry wt.	Swell Index	Unit Cohesion p.s.f.	Angle of Internal Friction degrees
7	3.5	111.1	17.7	0.2	700	25
8 Bag Sample	4-6			0.4	300 DISTURBED	24.5*
9	3.5	102.0	11.1			
9	8.5	79.8	15.7			
10	3.5	110.0	18.4			
10	8.5	112.0	17.8			
10	13.5	102.5	13.9	1.7	1300	15
11	3.5	106.0	14.2			

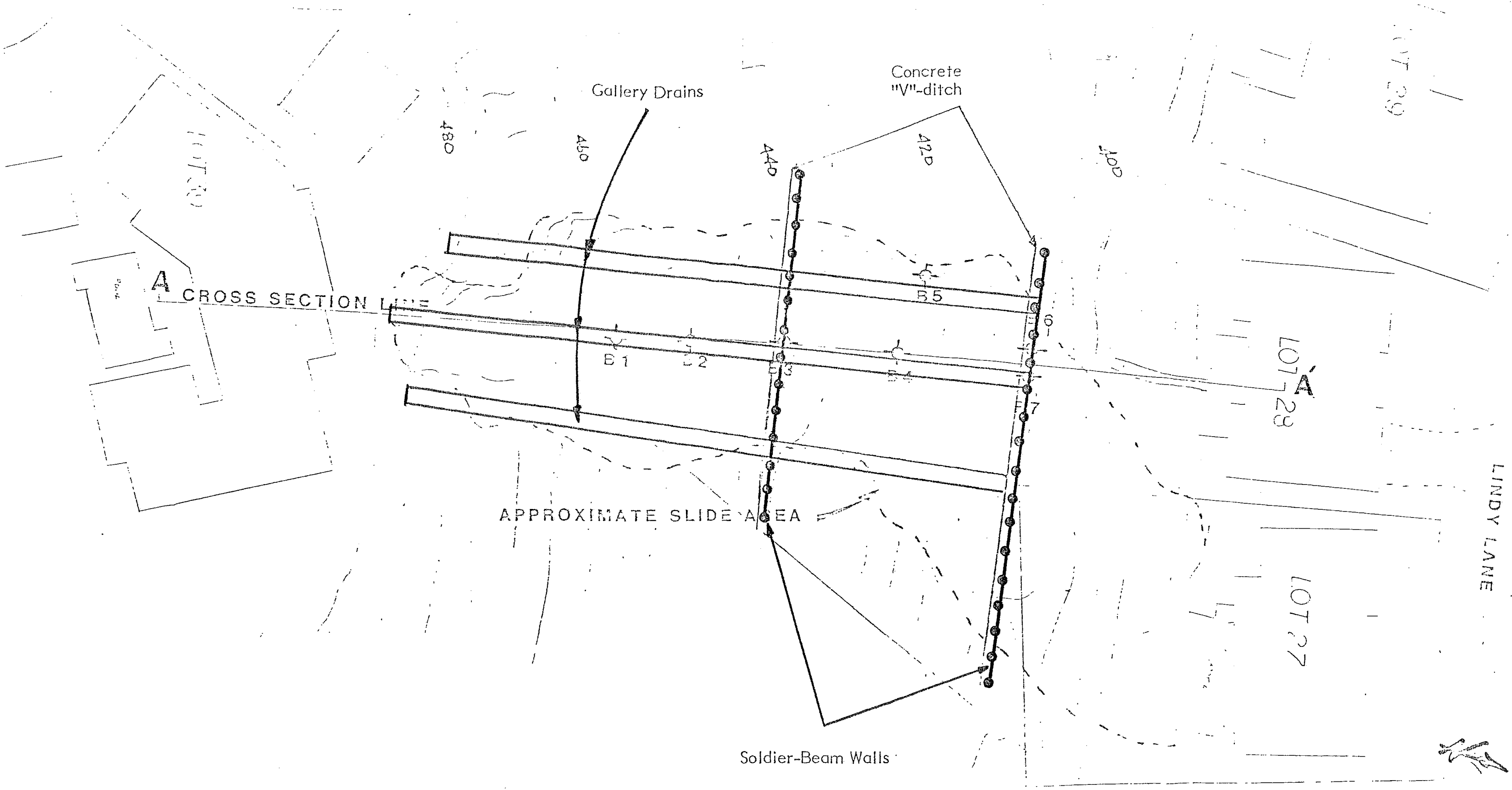
*Sample recompacted to 90% of maximum dry density as shown in Figure No. 8.

TABLE II

Summary of Hydrometer Analysis Test Results

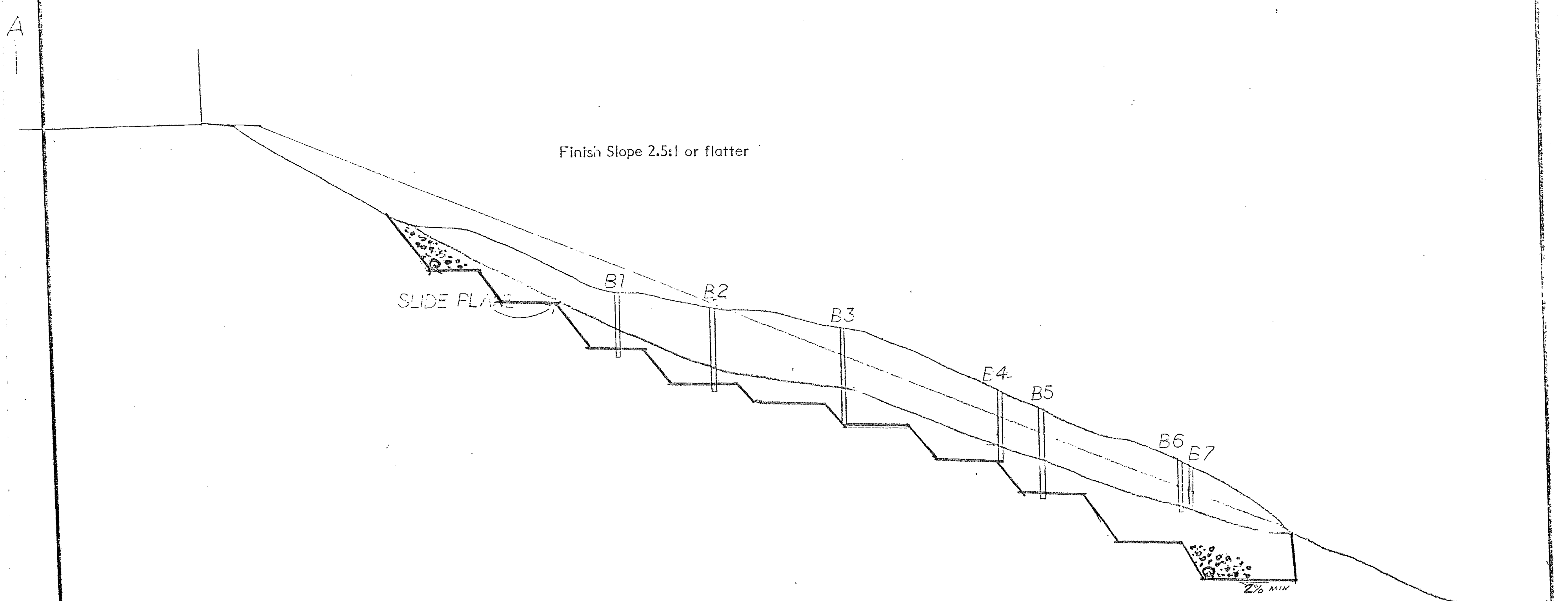
Hole No.	Depth ft.	Hydrometer Analysis Gravel %	Sand %	Silt %	Clay %
8	4-6	7	35	43	15

FIGURE 14: TEST RESULTS OF GRIBALDO, JACOBS & JONES (Continued)



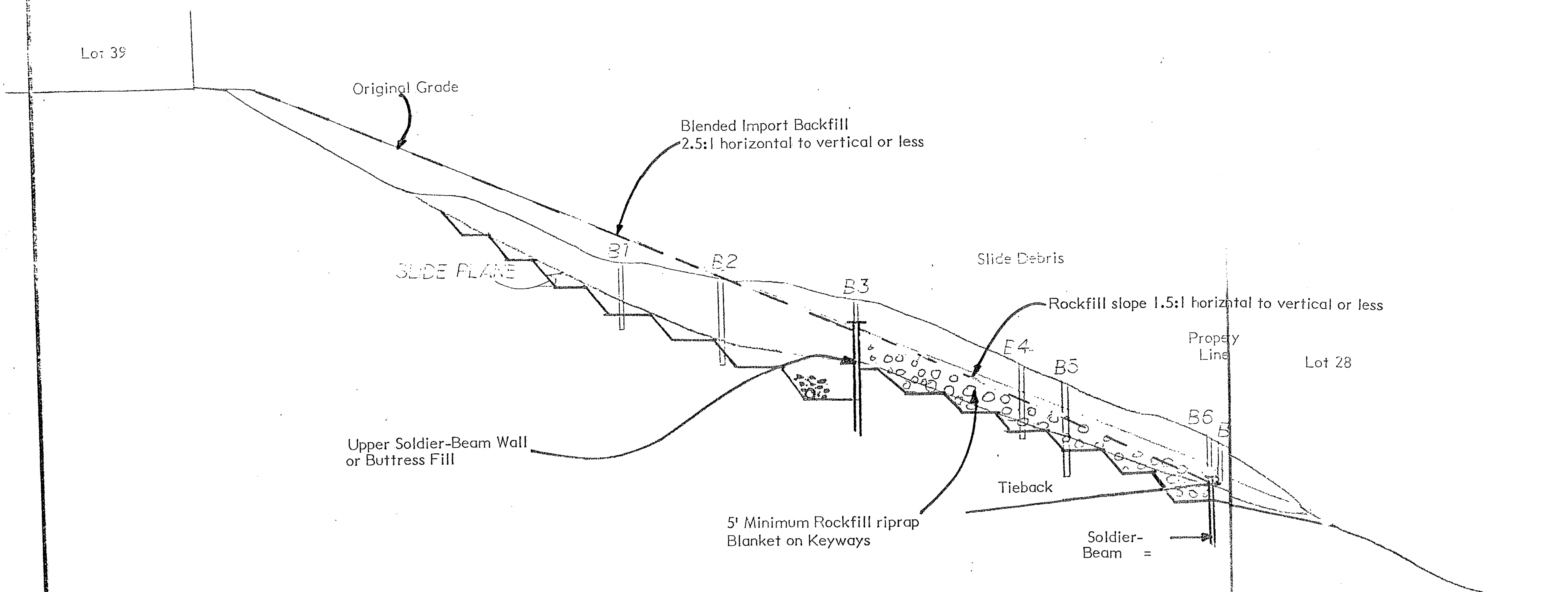
*Portion of Base Map Prepared by Frahm-Edler-Cannis, July 11, 1983, Job No. 3795

FIGURE 15	PURCELL, RHOADES & ASSOCIATES Foundation Engineering • Soil Engineering • Geology PLAN VIEW OF PROPOSED REPAIR FEATURES	
DATE 3/15/83	DRAWN BY IDA	CHECKED BY
SCALE 1" = 32' ±	W.O. 3179	



NOTE: Schematic only, not for bidding

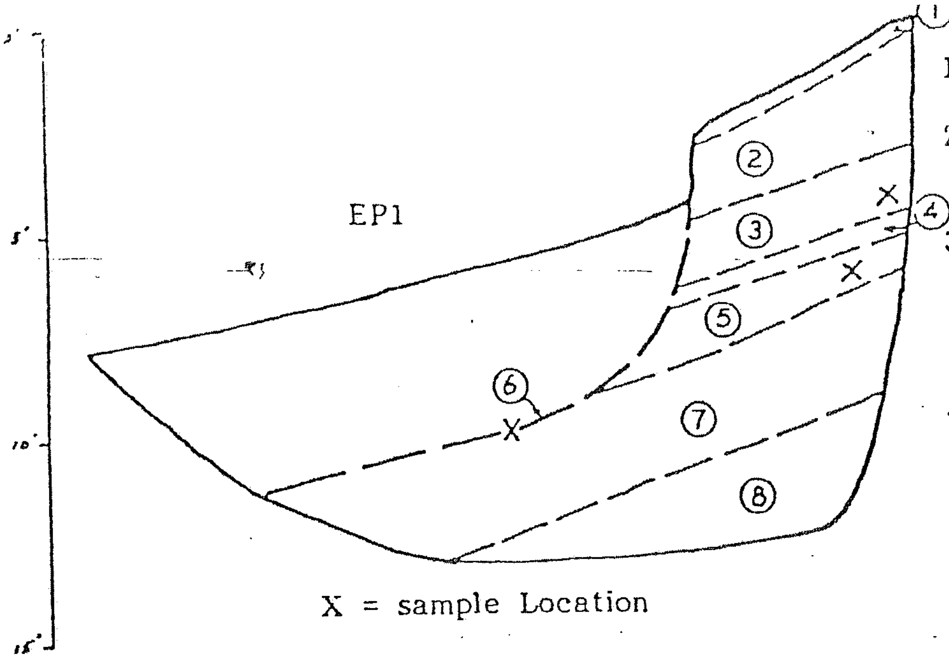
FIGURE 16	PURCELL, RHOADES & ASSOCIATES Foundation Engineering • Soil Engineering • Geology BUTTRESS FILL ALTERNATE
DATE 9/15/83	DRAWN BY IDA
CHECKED BY DJR	SCALE 1"=20' ±
W.O. 3179	



NOTE: Schematic only, not for bidding

FIGURE 17	PURCELL, RHOADES & ASSOCIATES Foundation Engineering • Soil Engineering • Geology SOLDIER-BEAM WALL ALTERNATE	
9/15/83	DRAWN BY IDA	CHECKED BY
1" = 20'	W.O. 3179	

EP1

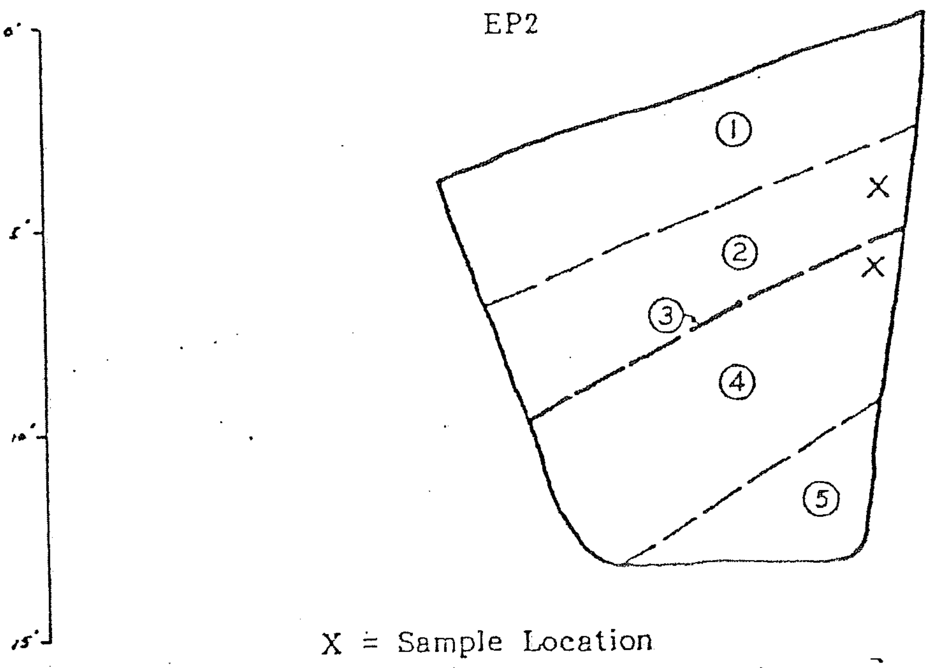


X = sample Location

1. SANDY CLAY, yellow-brown, (fill)
2. SANDY CLAY, dark brown, hard, drying cracks to 2' (soil profile)
3. GRAVELLEY SANDY CLAY, very stiff, moisture content = 25%, approx. 30° dip down slope(Weathered Santa Clara Formation)
4. CLAYEY SAND, yellow-brown, friable
5. SANDY CLAY, yellow-brown, very stiff
6. CLAY, wet, firm on slip surface, moisture content = 28%
7. SANDY CLAY, yellow-brown, slightly moist, very stiff
8. CLAYEY SANDSTONE, yellow-brown, fine sand (hard)

No. 5:
Moisture Content = 20%
Plasticity Index = 28%
Passing #200 sieve = 92%

EP2



X = Sample Location

1. SANDY CLAY, hard, 20% sand and fine gravel, (soil profile)
2. SANDY CLAY, yellow-brown, moist, stiff, 5% sand, Moisture Content = 29% Plasticity Index = 25% Passing #200 sieve = 96%
3. Slip surface, Clay seam, 1/8" thick, Dip = 25°
4. SANDY CLAY, yellow-brown, moist, very stiff, 20% sand
5. GRAVELLY CLAY, moist, very stiff, (Santa Clara Formation Conglomerate)

LOGS OF EXPLORATION PITS 1 AND 2

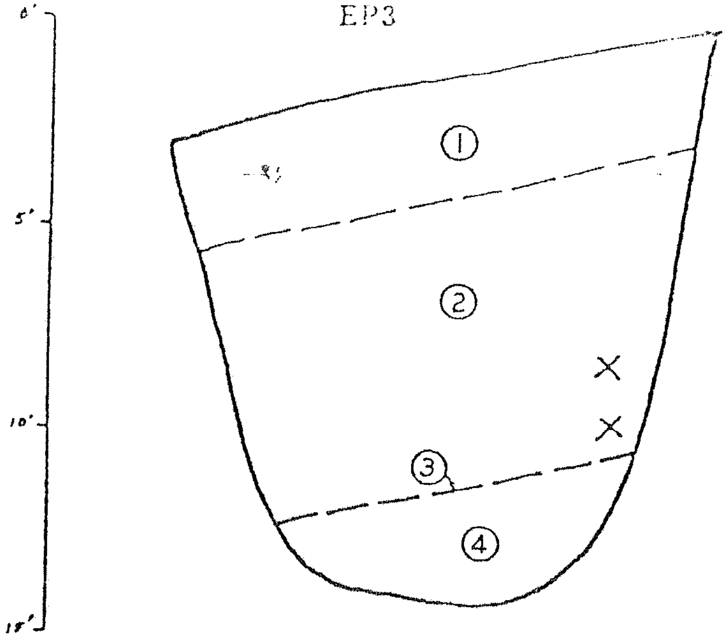
CLEARY CONSULTANTS, INC.
Geological and Geotechnical Engineers

LANDSLIDE INVESTIGATION
Lots 38 & 39, Tract 3354
Cupertino, California

APPROVED BY	SCALE	PROJECT NO.	DATE	DRAWING NO.
JMC	1" = 5'	317.1	Oct. 1982	6

FIGURE 18: CLEARY BORINGS

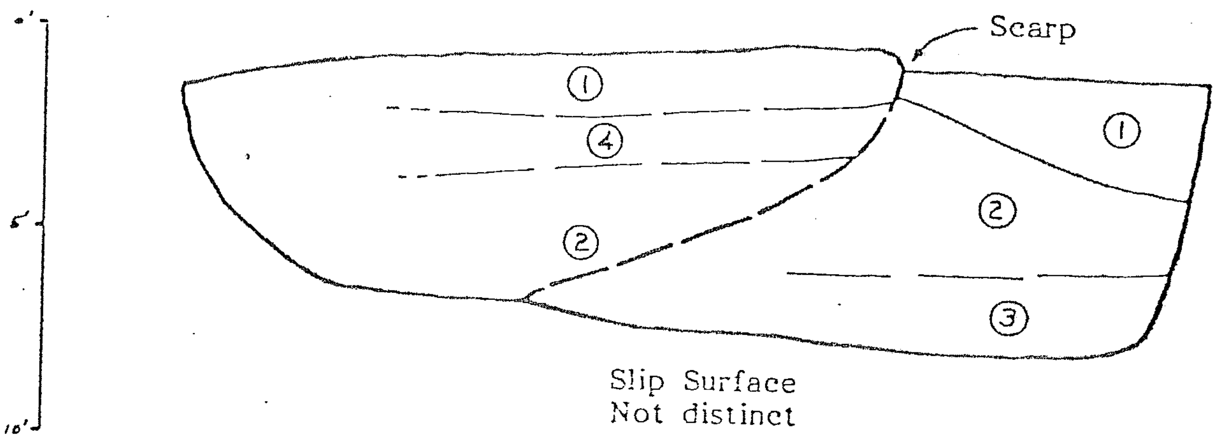
EP3



1. SILTY CLAY, dark brown, slightly moist, hard, drying cracks to 18" depth, soil profile
2. SANDY CLAY, moist, stiff, moisture content = 21%
3. Slip Surface, dip = 12°
4. CLAYSTONE, yellow-brown, very stiff, intact, slight seepage

X = Sample Locations

EP4



1. SANDY CLAY, dry, (fill)
2. GRAVELLY CLAY, brown, fine rounded gravel (Santa Clara Formation)
3. SANDY CLAY, slightly moist, very stiff
4. SANDY SILT, gray, dry, hard (residual soil)

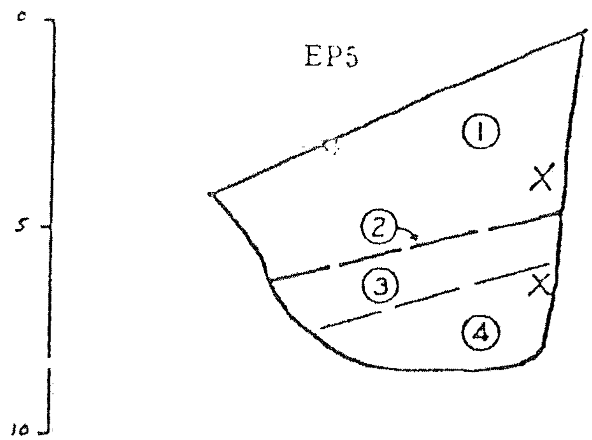
FIGURE 18: CLEARY BORINGS (Continued)

LOGS OF EXPLORATION PITS 3 and 4

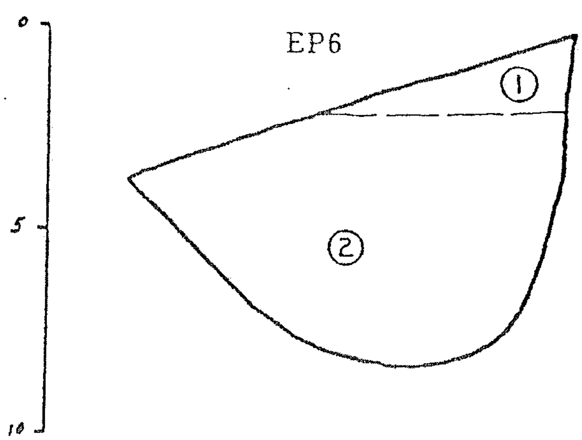
CCI CLEARY CONSULTANTS, INC.
Geological and Geotechnical Engineers

LANDSLIDE INVESTIGATION
Lots 38 & 39, Tract 3354
Cupertino, California

APPROVED BY	SCALE	PROJECT NO.	DATE	DRAWING NO.
JHC	1" = 5'	317.1	Oct. 1982	7



1. SANDY CLAY, moist, stiff, moisture content = 23%
2. Slip Surface
3. SANDY CLAY, moist, very stiff, dark brown
4. GRAVELLY CLAY and SANDY CLAY, yellow-brown, very stiff, (Santa Clara Formation) moisture content = 12%



1. SANDY CLAY, Light Brown, dry, hard (fill)
2. SANDY CLAY, Dark Brown, slightly moist, very stiff, (Weathered Santa Clara Formation)

FIGURE 18: CLEARY BORINGS (Continued)

LOGS OF EXPLORATION PITS 5 and 6

CLEARY CONSULTANTS, INC.
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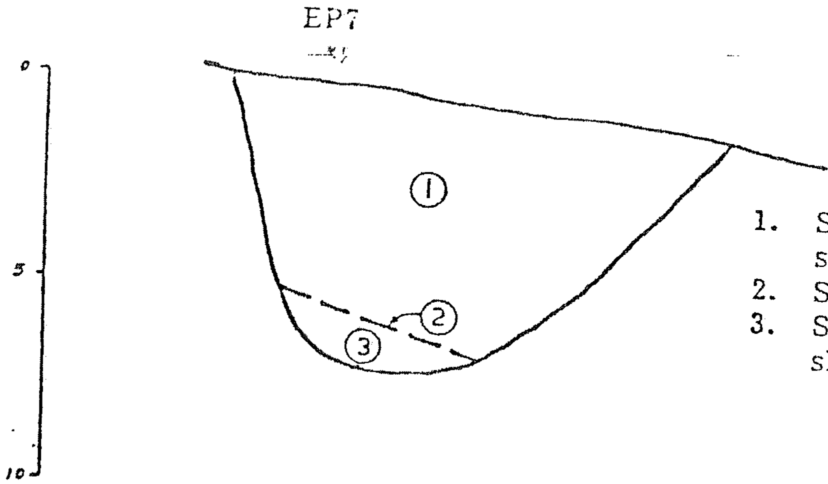
Lots 38 & 39, Tract 3354
Cupertino, California

APPROVED BY	SCALE	PROJECT NO.	DATE	DRAWING NO.
JMC	1" = 5'	317.1	Oct. 1982	8



1. SANDY CLAY, dark yellow-brown, slightly moist, stiff
2. Slip Surface
3. SANDY CLAY, dark yellow-brown, slightly moist, very stiff

FIGURE 18: CLEARY BORINGS (Continued)

LOG OF EXPLORATION PIT 7

CLEARY CONSULTANTS, INC.
Geological and Geotechnical Engineers

LANDSLIDE INVESTIGATION
Lots 38 & 39, Tract 3354
Cupertino, California

APPROVED BY	SCALE	PROJECT NO.	DATE	DRAWING NO.
JMC	1" = 5'	317.1	Oct. 1982	9

Catarina
I want to make certain that you are going to include the Rhodes and Purcell Soil report when you post on the website the material that is available for review.
Please respond to this request.

Larry W. Wilson
11446 Lindy Place
Cupertino CA 95014
408 255 3984
