LAW OFFICES OF
CARL B. METOYER

6014 Market Street
Oakland, CA 94608
(510) 658-1077

May 6, 2002

Henry Gannett
683 Ironbark Circle
Orinda, CA 94563

Re: Properties at 5712 and 5734
Country Club Drive, Oakland, CA

Dear Mr. Gannett:

Enclosed is a copy of the Geotechnical Engineers Report re the Replacement of The Failed Retaining Wall at my client's property located at 5712 Country Club Drive, Oakland, CA.

Please note the limitations on the use of the report set forth on Page 13.

Very truly yours,

CARL B. METOYER

CBM/cc
Encl.

REPORT
GEOTECHNICAL STUDY
Replacement of Failed Retaining Wall at 5712 Country Club Drive Oakland, California

March 29, 2002

Prepared for:

Dr. Vertis R. Thompson
5712 Country Club Drive
Oakland, California 94618

Prepared by:

GEOTECNIA
Consulting Geotechnical Engineers
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(925) 686-6556

Project Number: 020201

Luis E. Moura, C.E., G.E.
Principal

REGISTERED PROFESSIONAL ENGINEER
LUIS E. MOURA
2130
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EXPIRES
GEOTECHNICAL
STATE OF CALIFORNIA

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INTRODUCTION

Purpose

We completed a geotechnical study in association with the proposed replacement of a failed retaining wall along the north side of the residence at 5712 Country Club Drive in Oakland, California. The purposes of this site-specific study have been to evaluate the geotechnical conditions in the area just behind the failed retaining wall, evaluate the cause(s) of failure, and develop geotechnical criteria for design and construction of the proposed replacement retaining wall.

Scope of Services

The scope of our services was outlined in our Proposal and Professional Service Agreement dated February 7, 2002, and executed on February 13, 2002. Our study included performing a geotechnical reconnaissance of the site and immediate vicinity; reviewing selected geotechnical data and published geologic, landslide, and fault maps of the site vicinity; drilling, logging, and sampling three borings to depths of 8 to 16½ feet below the ground surface; performing laboratory tests on selected soil samples; conducting engineering analyses and geotechnical interpretations; and preparing this report.

This report contains the results of our study, including findings regarding site surface and subsurface conditions; conclusions pertaining to the cause(s) of wall failure and site-specific geotechnical conditions and geologic hazards; and geotechnical recommendations for wall stability analyses and design and construction of the proposed replacement retaining wall.

Pertinent exhibits appear in Appendix A. The site location relative to existing streets is shown on Plate 1 - Site Location Map. The locations of the borings are depicted relative to the retaining walls north of the existing residence on Plate 2 - Boring Location Map. The logs of the borings are displayed on Plates 3-5 - Logs of Borings B-1 through B-3. Explanations of the symbols and other codes used on the logs are presented on Plate 6 - Soil Classification Chart and Key to Test Data, and Plate 7 - Engineering Geology Rock Terms. The results of four Atterberg limits tests are presented on Plate 8 - Plasticity Chart. The lateral behavior criteria for drilled piers to support the alternative consisting of a soldier-pile-and-lagging wall are presented on Plate 9 - Drilled Pier Lateral Behavior. The recommended lateral pressures for design of the replacement wall are presented on Plate 10 - Retaining Wall Lateral Pressures. Plates 1-10 are included in Appendix A.

References consulted during the course of this study are listed in Appendix B. Details regarding the field exploration and laboratory testing programs appear in Appendix C.

Proposed Project/Background

Our understanding of the proposed project is based on conversations with the Client and the project structural engineer. We understand that the subject residence and surrounding retaining walls were constructed circa 1936. There are two retaining walls north of the subject residence: the lower, 4.5- to 6-foot-high retaining wall; and the upper, 5- to 6-foot-high retaining wall. An approximately 30-foot-long section of the lower retaining wall failed by virtue of the top of the wall rotating about 3 to 4 feet towards the north wall of the residence. This also caused tilting of a shed between the failed wall and the residence. Plate 2 shows the approximate locations of the retaining walls and tilted shed north of the house. The Client intends to replace the failed lower retaining wall. No other project details are known at this time.

FINDINGS

Site Description

The subject property is located at 5712 Country Club Drive, between Lincolnshire and Bowling Drives, in Oakland, California, as shown on Plate 1. The area immediately north of the subject residence includes a concrete patio and two retaining walls separating the subject property from the higher property to the north, as described above. The upper and lower retaining walls are only about two feet apart (from the assumed original location of the failed wall, as shown on Plate 2). The majority of the failed section of the lower wall is 6 feet high, although the wall becomes 4.5 feet high towards the west end. There were a few trees between the lower and upper retaining walls at the time of our field exploration program. The side yard on the adjacent property at 5734 Country Club Drive, behind the upper retaining wall, is relatively level and used as a garden.

The area to the east of the house also has a concrete patio and two retaining walls separating it from the higher property to the east. The upper retaining wall is cracked, and the top appears to have moved out about one foot towards the subject property. It appears that this wall is being held by a few trees growing on the subject property along its base. <u>The stability of the walls along the east side of the house was not part of the scope of this study</u>.

The area immediately west of the failed retaining wall includes the front lawn that slopes down towards the street. A large pine tree is located about 8 feet west of the end of the lower retaining wall, as shown on Plate 2.

Geology and Seismicity

The site is within the Coast Ranges Geomorphic Province, which includes the San Francisco Bay and the northwest-trending mountains that parallel the coast of California. These features were formed by tectonic forces resulting in extensive folding and faulting of the area. The oldest rocks in the area include sedimentary, volcanic, and metamorphic rocks of the Franciscan Complex, and sandstone, shale, and conglomerate of the Great Valley Sequence. These units are Jurassic to Cretaceous in age and form the basement rocks in the region.

The geologic map of the site vicinity (Radbruch, 1969) shows the site vicinity as being underlain by Jurassic and Cretaceous-age Franciscan shale and sandstone bedrock. The nearest active fault is the Type A Hayward Fault, located about 0.7 mile (1.2 kilometers) northeast of the site. The site is inside the two-kilometer, Near-Source Zone (NSZ) associated with the Hayward Fault (CDMG, 1997 and 1982).

Earth Materials

The subsurface conditions encountered in the three borings drilled for this study consisted of three separate horizons: a surficial layer of fill overlying native soil which, in turn, overlies shale and chert bedrock. The horizons encountered in the borings are described in more detail below in stratigraphic order starting at the ground surface. Detailed descriptions of the materials encountered in the three borings drilled for this study are shown on Plates 3-5.

The fill layers consisted of soft to medium stiff lean clay and sandy lean clay. The fill samples tested had dry densities ranging from 73 to 89 pounds per cubic foot (pcf), moisture contents ranging from 15.7 to 23.4 percent, Plasticity Indices (PIs) ranging from 17 to 25, and Standard Penetration Test (SPT) blow counts (N-values) ranging from about 3 to 8 blows per foot (bpf).

Underlying the fill in the three borings was a 1- to 2-foot-thick layer of stiff lean clay and sandy lean clay (native soil). The native soil samples tested had a dry density of 114 pcf, a moisture content of 12.0 percent, a PI of 12, pocket penetrometer shear strengths ranging from about 2,500 to 4,200 pounds per square foot (psf), and N-values ranging from about 8 to 18 bpf.

The bedrock encountered below the native soils in the three borings consisted of shale in Borings B-1 and B-3 and chert in Boring B-2. The depth to the top of the bedrock ranged from 6 feet in both Borings B-1 and B-3 to 8 feet in Boring B-2. The bedrock was soft to moderately hard, friable to moderately strong, and moderately highly weathered. The N-values in the bedrock ranged from about 20 bpf to 50 blows for 5 inches of penetration.

Groundwater

No free groundwater was encountered in any of the three borings drilled for this study. The groundwater level is anticipated to fluctuate with changes in seasonal and annual precipitation, irrigation, and other factors.

CONCLUSIONS

General

Based on the results of this study, it is our opinion that the subject retaining wall may have failed due to a combination of factors as described below. In order to determine the predominant cause of the failure, additional information would be required, including the exact size of the footing (depth, width, and thickness), any information regarding the design of the wall, the date of the failure, the environmental/weather conditions immediately before and at the time of the failure, and whether the failure was a progressive or sudden failure. The primary geotechnical/geologic considerations associated with design and construction of the proposed replacement retaining wall are (1) the presence of weak, expansive, and creeping clay backfill soils; (2) maintaining the stability of the upper retaining wall during construction of the replacement wall; and (3) seismic shaking during earthquakes. These items are addressed in greater detail below.

Causes of Failure of Lower Retaining Wall

Based on the results of this study, we have identified several factors that may have contributed to the failure of the lower retaining wall. It appears that the failure was either an overturning or stem failure. We did not encounter any concrete in Boring B-1, which was drilled right behind the assumed original location of the wall (see Plate 2). This suggests that if a footing heel is present, it must be very short. As discussed above, additional information would be required to determine which cause or causes were the predominant causes of failure. In our opinion, the potential causes of failure were:

- Long-term lateral earth pressures that may be significantly higher than the design pressures;
- The lateral surcharge pressure from the upper retaining wall;
- The effect of the roots and dynamic wind loads from the trees behind the wall and large pine tree west of the wall;
- Poor drainage behind the wall;
- Seismic surcharge pressures; and
- Poor construction, materials failure, or age of wall.

The long-term lateral earth pressure developed by the moderately expansive soils behind the retaining wall (with PIs ranging from 17 to 25) can reach values that were probably 2 to 3 times higher than the design lateral pressure, in our opinion. Cantilever retaining walls like the failed wall were typically designed at the time the wall was constructed for lateral earth pressures based on equivalent fluid weights ranging from about 35 to 50 pcf. Although initially the design pressure may actually have been greater than the actual pressure, over time the pressure increased substantially due to creeping of the expansive clays towards the back of the wall. Over the long-term, it is our opinion that the actual equivalent fluid weight was about equal to the average unit (wet) weight of the backfill soils, or about 105 pcf. This value is about 2 to 3 times greater than the typical design values given above. Although we do not know the actual design value or the factor of safety used, it is our opinion that this was probably one of the predominant causes of the failure.

The calculated lateral surcharge from the upper retaining wall above the level of the patio slab is about 1,500 pounds per foot of wall, for the 6-foot-high wall (see Plate 10). Since we do not know whether a surcharge pressure was used in the design of the wall, we cannot comment on whether a surcharge was used, or whether the surcharge used was appropriate.

The trees growing between the lower and upper retaining walls, as well as the large pine tree located about 8 feet from the west end of the failed wall, contributed negatively towards the stability of the lower wall in two ways. First, as the trees grew, their root systems exerted more and more pressure against the back of the wall. Secondly, as the trees continued to grow, wind loads on the canopies were transferred to the wall as large driving moments that reduced the stability of the wall. Again, since we do not have information regarding the relationship between the weather and the time of the failure, we cannot comment on the exact contribution of the trees towards the failure. However, we recommend below that (1) the trees be removed from the area between the failed wall and the upper retaining wall, and (2) the roots from the large pine tree extending below the retaining wall be removed.

Poor drainage behind the wall may also have contributed to the failure by increasing the lateral pressure due to the effect of hydrostatic pressures developing behind the wall. Further exploration behind the wall would be required to determine whether a backdrain is present and whether it was functioning (i.e. not clogged).

The final probable causes from a geotechnical viewpoint, in our opinion, were potential seismic surcharge pressures during strong seismic shaking at the site since construction of the wall. Since the wall was constructed, the largest seismic event in the Bay Area was the 1989 Loma Prieta earthquake. Based on published studies of ground surface accelerations during that earthquake (Plafker and Galloway, 1989), the accelerations in the general site vicinity ranged from about 0.08 to 0.29 g; however, for rock sites, which better represent the subject site, the accelerations were probably between 0.08 and 0.16 g. This may have caused surcharges behind the wall ranging from about 100 to 350 pounds per foot of wall, and may have contributed toward the failure; however, we would need further information on the timing of the failure to be able to comment on the extent seismic surcharge pressures may have contributed to the failure.

The final three potential causes listed above (poor construction, materials failure, or age of the wall) were grouped together since they are not geotechnical issues but rather structural engineering issues. We suggest that the structural engineer evaluate these potential causes.

We would be available to help perform stability analyses of the failed wall once more information is available regarding the footing dimensions.

Presence of Weak, Expansive, and Creeping Clay Backfill Soils

The results of our field exploration and laboratory testing program indicate that the fill soils consist of soft to medium stiff clays with relatively low densities and a moderate potential for expansion. The potential for expansion is tabulated below as a function of the PI. As shown on the table, the fill materials behind the failed wall (with PIs of 17 to 25) have a moderate potential for expansion.

Approximate PI Range	Expansion Potential
<12	Nil
12-15	Low
15-25	Moderate
25-35	High
>35	Very High

When expansive soil behavior occurs on slopes or behind retaining walls, such as at the subject site, there is a component of movement parallel to the downslope direction. Slope creep is a slow process, typically involving a fraction of an inch per year; however, this movement accumulates over the years and can result in several inches of lateral movement over the life of a structure. When a retaining wall prevents these movements from occurring, the result is that the actual pressures against the wall increase with time due to the creep. The recommended earth pressure given below for design of the replacement retaining wall is the long-term pressure assuming that creep will occur over the life of the replacement wall, in our opinion.

Stability of Upper Retaining Wall

During construction of the replacement of the lower retaining wall, the stability of the upper retaining wall should be addressed. The factor of safety against failure of that wall is greater than one since the wall is still standing; however, the required excavation during construction of the replacement lower wall may reduce that factor of safety to less than one and cause the upper wall to fail.

In order to evaluate the stability of the upper retaining wall during construction of the replacement lower wall, additional information is required regarding the depth and size of the footing supporting it. This may require pot-holing if no plans are available due to the age of that wall.

If the stability of the upper wall cannot be properly evaluated, the construction method or type of replacement wall should be designed with specific measures to maintain the stability of the upper wall. A soldier-pile-and-lagging wall could be built between the two walls by first installing the soldier piles so that as excavation proceeds downward, lagging between the soldier piles would be placed to hold the backfill in place. Recommendations are presented below for both a footing-supported cantilever wall and for a soldier-pile-and-lagging wall as the two types of replacement walls to be considered.

Other Geologic Hazards

It is our opinion that the potentials for liquefaction, lateral spreading, and seismic compaction are low at the location of the replacement wall because no loose, saturated granular soils were encountered in the three borings completed for this study. The potentials for landsliding, fault rupture, and earthquake shaking are discussed below.

Landsliding

Published geologic and slope stability maps of the site vicinity reviewed for this study did not show landslides at the site or its immediate vicinity (Nilsen, 1975; Radbruch, 1969), and the site is not in a zone classified as susceptible to earthquake-induced landsliding (CDMG, 2000). During our site reconnaissance, we did not observe evidence of deep-seated, active slope instability at the site or its immediate vicinity.

Although landsliding is unlikely at the site, there are some inherent risks associated with buildings on slopes in seismic areas such as the San Francisco Bay Area. The owner should be aware of these risks associated with the subject property, which could include seismically induced slope movements. Due to the proximity of the site to the active Hayward Fault, ground shaking at the site could be strong during a large earthquake occurring on a nearby segment of that fault as discussed below.

Fault Rupture

The subject property does not lie within the Alquist-Priolo Earthquake Fault Zone associated with the Type A Hayward Fault, as defined by the California Division of Mines and Geology. No faults are shown crossing the site on reviewed published maps, nor did we observe evidence of faulting during our study. Therefore, we conclude that the potential risk for damage to the planned improvements at the site due to surface rupture from faults is low.

Earthquake Shaking

Earthquake shaking results from the sudden release of seismic energy during displacement along a fault. During an earthquake, the intensity of ground shaking at a particular location will depend on a number of factors including the earthquake magnitude, the distance to the zone of energy release, and local geologic conditions. We expect that the site may be exposed to strong earthquake shaking during the life of the proposed replacement wall since the site is inside the NSZ associated with the Hayward Fault. The recommendations contained in the latest enforced version of the Uniform Building Code (UBC) should be followed for reducing potential damage to the structure from earthquake shaking.

Static Loads

The replacement retaining wall should be designed to resist a static lateral earth pressure equivalent to that exerted by a fluid weighing 105 pcf, assuming level backfill conditions. This value is the same for either active or at-rest conditions.

In addition to the lateral earth pressure, the retaining wall must also be designed to resist horizontal pressures that may be generated by surcharge loads applied at or near the ground surface. The lateral surcharge pressure distribution from the upper retaining wall is presented on Plate 9. For different types of surcharge loads, we can provide the appropriate lateral surcharge pressures on the retaining wall once the geometry and loading conditions are defined.

Seismic Loads

Horizontal accelerations during seismic events will momentarily increase lateral earth pressures against walls. We recommend using an equivalent seismically induced earth pressure with a rectangular distribution equal to FH psf, where F depends on the magnitude of the ground acceleration and H is the unsupported (free) wall height in feet. The resultant seismic force would act at 0.5H above the base of the wall. The seismic earth pressures are in addition to the static earth pressure and lateral surcharge pressures, and should be considered in the design of the replacement retaining wall.

The anticipated peak ground surface acceleration (pga) at the site during the maximum credible earthquake on the Hayward Fault is estimated to be about 0.7 g. Using an estimated pga of 0.7 g at the site, the value of F would be 25. For smaller seismic events, the value of F would be lower. For a typical design pga of 0.2 to 0.3 g for residential structures, the value of F would be between 7 and 10. The choice of the value of F to be used for retaining wall design depends on the level of risk accepted by the designer and owner. If the wall is not designed for the appropriate seismically induced earth pressures, consequences during strong earthquake loading might include lateral movement, distress, or failure of the wall.

The magnitudes of the seismically-induced earth pressures above were calculated based on the simplified procedure developed by Seed and Whitman (1970) and incorporated a reduction factor on the order of 20 percent to judgmentally account for possible effects of wave scattering or passage, the transient nature of earthquake ground motions, and possible wall-soil interaction effects.

Retaining Wall Backdrains

The retaining wall should be fully backdrained. The backdrain may consist of a prefabricated drainage structure provided our firm is given the opportunity to review the manufacturer's details. Alternately, a conventional backdrain consisting of a 4-inch-diameter, rigid perforated pipe surrounded by a drainage blanket may be used. The pipe should be sloped to drain by gravity to appropriate outlets. The drainage blanket should consist of Caltrans Class 2 "Permeable Material." Alternately, the drainage blanket could consist of clean, free-draining crushed rock or gravel, wrapped in a filter fabric such as Mirafi 140N. The top of the drainpipe should be at least 8 inches below the lowest adjacent grade. The drainage blanket in a conventional backdrain should be at least one foot wide and extend to within one foot of the surface, and the uppermost one-foot should be backfilled with compacted in-situ soils to exclude surface water.

Accessible subdrain cleanouts should be provided and maintained on a routine basis. The drainage facilities should be cleaned and maintained as necessary so that they continue to function properly. All collected drainage should be discharged through closed conduits into the storm drain system.

Supplemental Services

We recommend that GEOTECNIA be retained to review the project plans, specifications, and structural calculations to evaluate if they are in general conformance with the intent of our geotechnical recommendations. In addition, we should be retained to observe geotechnical construction, particularly site excavations, footing excavations, drilled pier construction, placement of subsurface drainage behind retaining walls, subgrade preparation, fill and backfill placement and compaction, and to perform appropriate field and laboratory testing.

If, during construction, subsurface conditions different from those encountered in the exploratory borings are observed, or appear to be present beneath excavations, we should be advised at once so that these conditions may be reviewed and our recommendations reconsidered. The recommendations made in this report are contingent upon our notification and review of the changed conditions.

If more than 18 months have elapsed between the submission of this report and the start of work at the site, or if conditions have changed because of natural causes or construction operations at or adjacent to the site, the recommendations of this report may no longer be valid or appropriate. In such case, we recommend that we review this report to determine the applicability of the conclusions and recommendations considering the time elapsed or changed conditions. The recommendations made in this report are contingent upon such a review.

These services are performed on an as-requested basis and are in addition to this geotechnical study. We cannot accept responsibility for conditions, situations, or stages of construction that we are not notified to observe.

LIMITATIONS

This report has been prepared for the exclusive use of the owner (Dr. Vertis R. Thompson), the project structural engineer (Mr. Joseph Oakley, Jr.), as well as their agents or consultants, for the proposed project described in this report. The recommendations in this report should not be applied to structures or locations other than those described in this report.

Our services consist of professional opinions and conclusions developed in accordance with generally accepted geotechnical engineering principles and practices. We provide no other warranty, either expressed or implied. Our conclusions and recommendations are based on the information provided us regarding the proposed construction, review of available data, the results of our field exploration and laboratory testing programs, and professional judgment. Verification of our conclusions and recommendations is subject to our review of the project plans, specifications, and structural calculations, and our observation of construction.

The boring logs represent subsurface conditions at the location and on the date indicated. It is not warranted that they are representative of such conditions elsewhere or at other times. Site conditions and cultural features described in the text of this report are those existing at the time of our field exploration, conducted on March 11, 2002, and may not necessarily be the same or comparable at other times

APPENDIX B

List of References

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2. California Division of Mines and Geology, 1997, *Active Fault Near-Source Zones,* Department of Conservation, Sheet E-17, Scale ¼ inch = 1 kilometer.

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5. GEOTECNIA, 2001, *Report, Pavement Geotechnical Study, Glenview Elementary School, 4215 La Cresta Avenue, Oakland, California,* prepared for Ackland International, Inc., Job No. 010301, dated May 7.

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7. Jennings, C.W., 1994, *Fault Activity Map of California and Adjacent Areas,* California Department of Conservation, Division of Mines and Geology, Geologic Data Map No. 6.

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14. Seed, H.B., and Whitman, R.V., 1970, *Design of Earth Retaining Structures for Dynamic Loads*, Proceedings of the Specialty Conference on Lateral Stresses and Earth Retaining Structures, Soil Mechanics and Foundations Division, American Society of Civil Engineers.

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APPENDIX C

Field Exploration

Our field exploration included a geologic reconnaissance and subsurface exploration program consisting of drilling and sampling three borings behind the failed retaining wall north of the subject residence on March 11, 2002. The borings were drilled and sampled at the approximate locations shown on Plate 2 using portable flight auger and sampling equipment. One of the borings (B-2) was drilled behind the upper retaining wall, on the side yard of the adjacent residence at 5734 Country Club Drive.

The logs of the borings are displayed on Plates 3-5. Representative disturbed and relatively undisturbed samples of the earth materials were obtained from the borings at selected depth intervals with a 2-inch-diameter, split-barrel Standard Penetration Test (SPT) sampler and a 3-inch-diameter, modified California sampler, respectively.

Penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration. These blow counts were then correlated to SPT blow counts. The blows per foot recorded on the Boring Logs represent the accumulated number of blows (correlated to SPT blow counts) that were required to drive the sampler the last 12 inches or fraction thereof. A correction factor of 2/3 was applied to the field blow counts for the modified California sampler.

The soil and bedrock classifications are shown on the Boring Logs and referenced on Plates 6 and 7.

Laboratory Testing

Natural water contents and dry densities were determined for selected samples recovered from the borings. The data from these tests are recorded at the appropriate sample depths on the boring logs. Four Atterberg limits tests were performed on the clayey soil samples collected from Borings B-1 and B-2 to evaluate the potential for expansion of those soils. The results of these tests are also presented on the boring logs and on Plate 8.

APPENDIX D

Distribution

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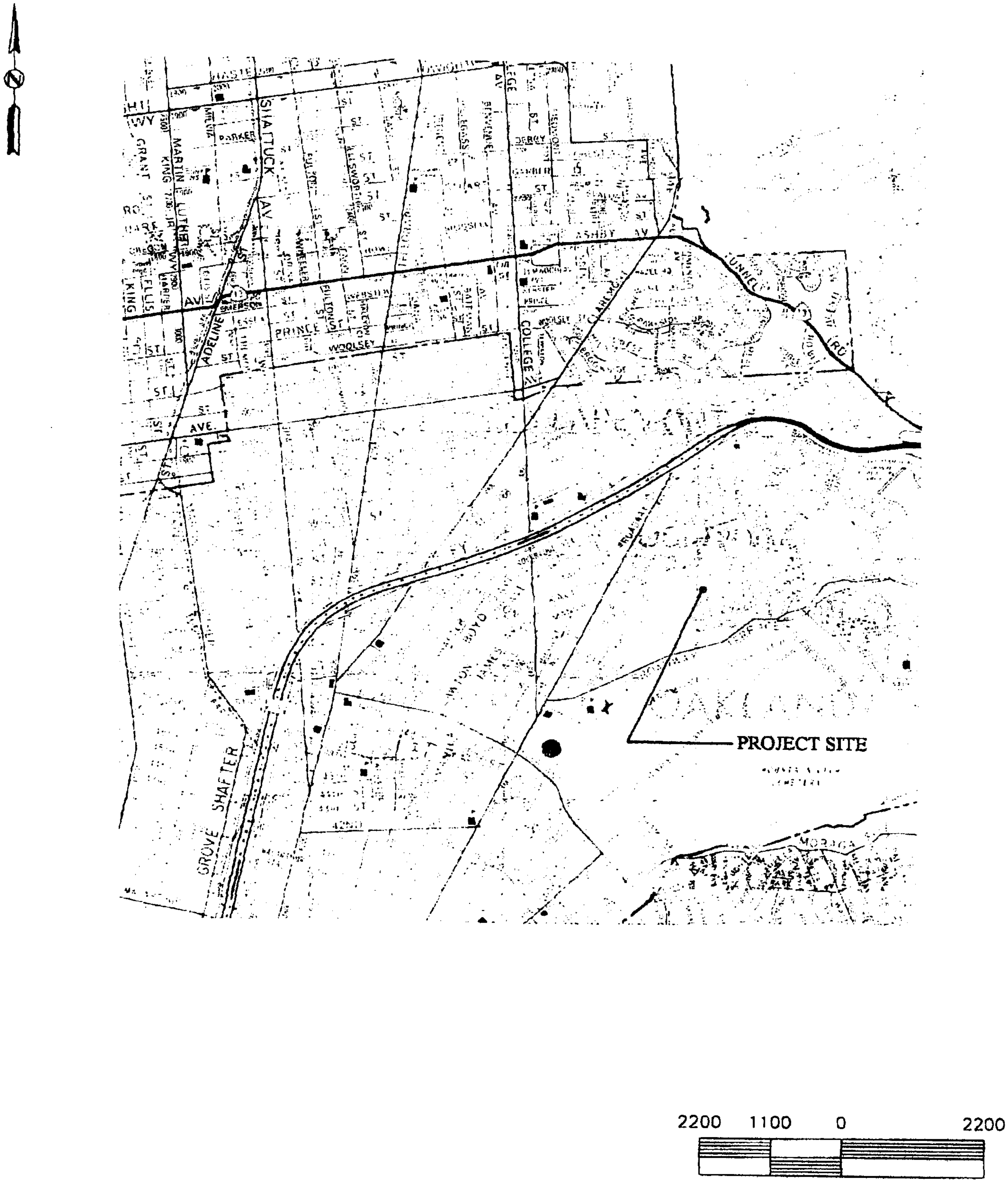

Reference: Thomas Bros. Maps, Alameda County.

GEOTECNIA Consulting Engineers	Job. No: 020201 Appr: [signature] Drwn: CD Date: MAR 2002	SITE LOCATION MAP Failed Retaining Wall at 5712 Country Club Drive Oakland, California	PLATE 1

EQUIPMENT: **3.5-inch Flight Auger** ELEVATION:

LOGGED BY: **LEM** START DATE: **3-11-02**

FINISH DATE: **3-11-02**

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Depth (feet)	Description
					3	0	SANDY LEAN CLAY (CL), mottled brown, soft, moist, with rock fragments
		23.4	82				
LL = 42, PI = 17; see Plate 8						1	
					4	2	
					14	3	
LL = 48, PI = 25; see Plate 8		21.4	89				
	2.5					4	FILL
							SANDY LEAN CLAY (CL), grey-brown, stiff, moist
					16		
LL = 35, PI = 12; see Plate 8		12.0	114			5	
	4.0						
					47	6	SHALE, firm, weak, highly weathered
	>4.5					7	
						8	
					48	9	
	>4.5					10	
						11	
						12	
						13	
						14	
					70	15	
	>4.5					16	

* Converted to equivalent standard penetration blow counts.

Bottom of Boring B-1 at a depth of 16.5 feet.
No free groundwater encountered at time of drilling.

GEOTECNIA
Consulting Engineers

Job No: 020201
Appr: [signature]
Drwn: CD
Date: MAR 2002

LOG OF BORING B-1

Failed Retaining Wall at
5712 Country Club Drive
Oakland, California

PLATE
3

EQUIPMENT: 3.5-inch Flight Auger
ELEVATION:
LOGGED BY: LEM
START DATE: 3-11-02
FINISH DATE: 3-11-02

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	Description
							0	LEAN CLAY (CL), dark brown, medium stiff, moist, with rock fragments
		15.7	73		8		1–2	
LL = 50, PI = 26; see Plate 8	3.2				11		6–7	FILL
							7	SANDY LEAN CLAY (CL), grey-brown, stiff, moist
	>4.5				50/5"		8	CHERT, moderately hard, moderately strong, moderately weathered
							9	Bottom of Boring B-2 at a depth of 9 feet. No free groundwater encountered at time of drilling.

* Converted to equivalent standard penetration blow counts.

GEOTECNIA
Consulting Engineers

Job No: 020201
Appr: [signature]
Drwn: CD
Date: MAR 2002

LOG OF BORING B-2

Failed Retaining Wall at
5712 Country Club Drive
Oakland, California

PLATE 4

EQUIPMENT: Continuous Sampling

LOGGED BY: LEM

ELEVATION:

START DATE: 3-11-02

FINISH DATE: 3-11-02

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	DEPTH (FEET)	Description
					4	0	LEAN CLAY (CL), dark brown, medium stiff, moist, with roots
	3.5					1	
					7	2	
	3.2						
						3	
					7		
	2.7					4	
							FILL
							LEAN CLAY (CL), grey-brown, stiff, moist, with roots
	4.2					5	
					21		
						6	SHALE, soft, friable, highly weathered
					20		
						7	
						8	Bottom of Boring B-3 at a depth of 8 feet. No free groundwater encountered at time of drilling.

* Converted to equivalent standard penetration blow counts.

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Date: MAR 2002

LOG OF BORING B-3

Failed Retaining Wall at
5712 Country Club Drive
Oakland, California

PLATE 5

MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS More than Half > #200 sieve	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS
			SP	POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, POOORLY GRADED SAND-SILT MIXTURES
			SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS More than Half < #200 sieve	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS

UNIFIED SOIL CLASSIFICATION SYSTEM

Shear Strength, psf
Confining Pressure, psf

Consol	Consolidation	Tx	2630 (240)	Unconsolidated Undrained Triaxial
LL	Liquid Limit (in %)	Tx sat	2100 (575)	Unconsolidated Undrained Triaxial, saturated prior to test
PL	Plastic Limit (in %)	DS	3740 (960)	Unconsolidated Undrained Direct Shear
PI	Plasticity Index	TV	1320	Torvane Shear
Gs	Specific Gravity	UC	4200	Unconfined Compression
SA	Sieve Analysis	LVS	500	Laboratory Vane Shear
■	Undisturbed Sample (2.5-inch ID)	FS	Free Swell	
◪	2-inch-ID Sample	EI	Expansion Index	
◩	Standard Penetration Test	Perm	Permeability	
⊠	Bulk Sample	SE	Sand Equivalent	

KEY TO TEST DATA

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SOIL CLASSIFICATION CHART AND KEY TO TEST DATA
Failed Retaining Wall at
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Oakland, California

PLATE 6

ROCK SYMBOLS

SHALE OR CLAYSTONE	CHERT	SERPENTINITE
SILTSTONE	PYROCLASTIC	METAMORPHIC ROCKS
SANDSTONE	VOLCANIC	DIATOMITE
CONGLOMERATE	PLUTONIC	SHEARED ROCKS

LAYERING

MASSIVE	Greater than 6 feet
THICKLY BEDDED	2 to 6 feet
MEDIUM BEDDED	8 to 24 inches
THINNLY BEDDED	2-1/2 to 8 inches
VERY THINNLY BEDDED	3/4 to 2-1/2 inches
CLOSELY LAMINATED	1/4 to 3/4 inches
VERY CLOSELY LAMINATED	Less than 1/4 inch

JOINT, FRACTURE, OR SHEAR SPACING

VERY WIDELY SPACED	Greater than 6 feet
WIDELY SPACED	2 to 6 feet
MODERATELY SPACED	8 to 24 inches
CLOSELY SPACED	2-1/2 to 8 inches
VERY CLOSELY SPACED	3/4 to 2-1/2 inches
EXTREMELY CLOSELY SPACED	Less than 3/4 inch

HARDNESS

SOFT - Pliable; can be dug by hand

FIRM - Can be gouged deeply or carved with a pocket knife

MODERATELY HARD - Can be readily scrached by a knife blade; scratch leaves heavy trace of dust and is readily visable after the powder has been blown away

HARD - Can be scratched with difficulty; scratch produces little powder and is often faintly visable

VERY HARD - Cannot be scratched with pocket knife; leaves a metallic streak

STRENGTH

PLASTIC - Capable of being molded by hand

FRIABLE - Crumbles by rubbing with fingers

WEAK - An unfractured specimen of such material will crumble under light hammer blows

MODERATELY STRONG - Specimen will withstand a few heavy hammer blows before breaking

STRONG - Specimem will withstand a few heavy ringing hammer blows and usually yields large fragments

VERY STRONG - Rock will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments

DEGREE OF WEATHERING

HIGHLY WEATHERED - Abundant fractures coated with oxides, carbonates, sulphates, mud, etc., thourough discoloration, rock disintegration, mineral decomposition

MODERATELY WEATHERED - Some fracture coating, moderate or localized discoloration, little to no effect on cementation, slight mineral decomposition

SLIGHTLY WEATHERED - A few stained fractures, slight discoloration, little or no effect on cementation, no mineral decomposition

FRESH - Unaffected by weathering agents, no appreciable change with depth

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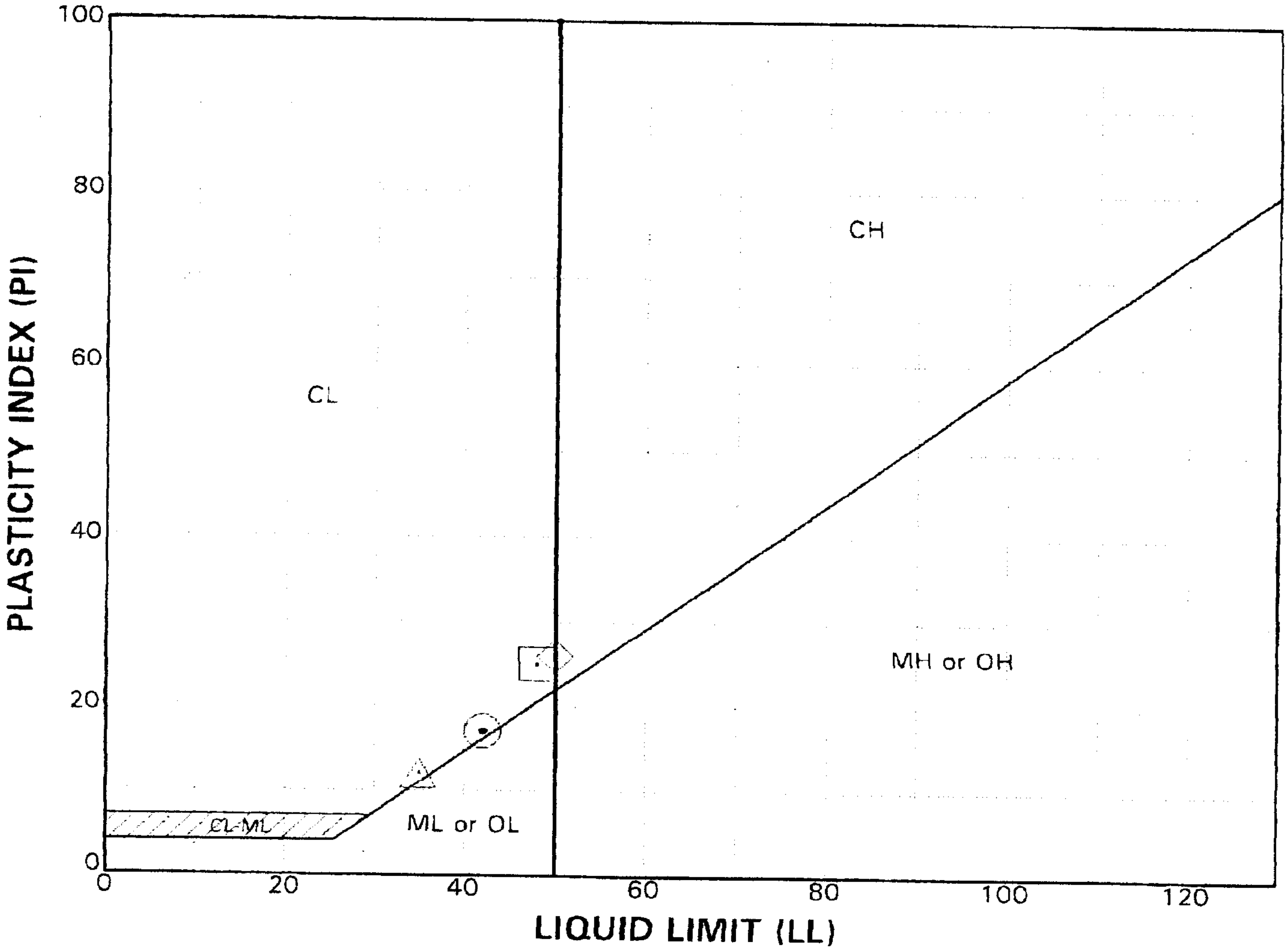
Date: MAR 2002

ENGINEERING GEOLOGY ROCK TERMS

Failed Retaining Wall at
5712 Country Club Drive
Oakland, California

PLATE 7



SAMPLE SOURCE	CLASSIFICATION	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)	% PASSING #200 SIEVE
⊙ Bor. B-1 @ 1.0'	Sandy Lean Clay (CL)	42	25	17	
⊡ Bor. B-1 @ 3.5'	Sandy Lean Clay (CL)	48	23	25	
△ Bor. B-1 @ 5.0'	Sandy Lean Clay (CL)	35	23	12	
◇ Bor. B-2 @ 6.5'	Lean Clay (CL)	50	24	26	

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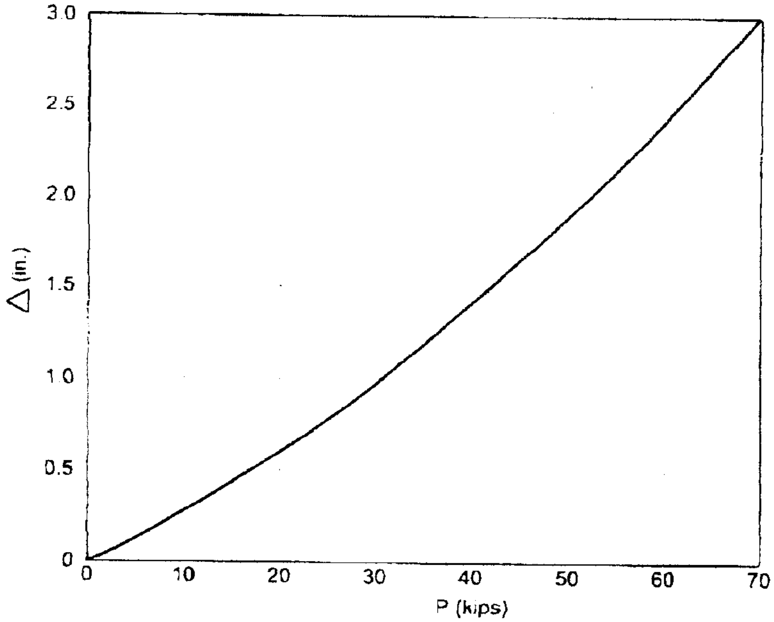
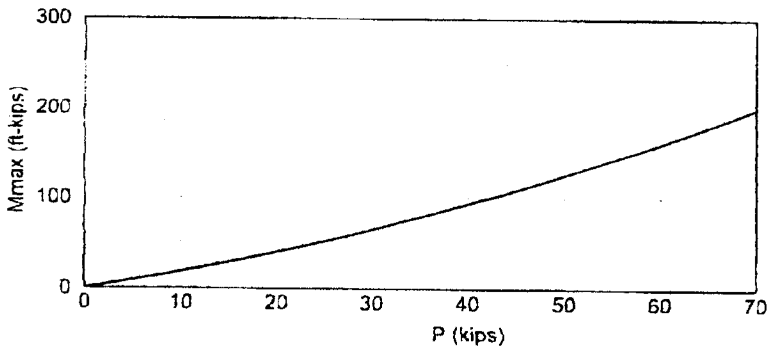
Date: MAR 2002

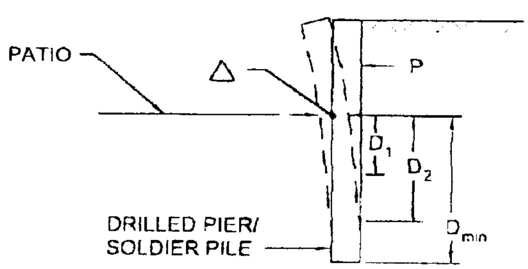
PLASTICITY CHART

Failed Retaining Wall at
5712 Country Club Drive
Oakland, California

PLATE
8

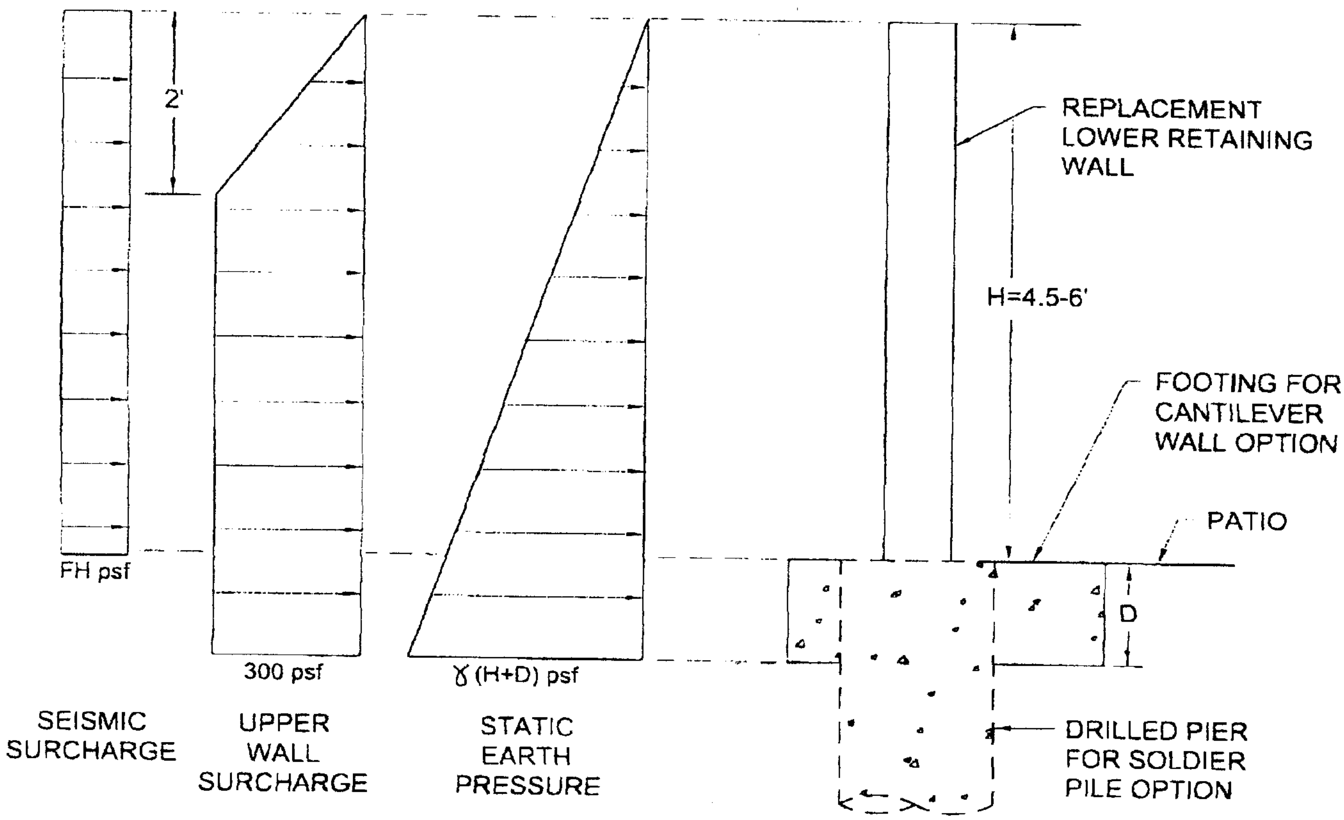






P = Total lateral load on wall
Δ = Pier deflection at top of patio slab below wall
Mmax = Maximum moment
D_1 = Depth to Mmax (4 ft)
D_2 = Depth to first zero deflection (7 ft)
D_{min} = Minimum pier depth (9 ft)

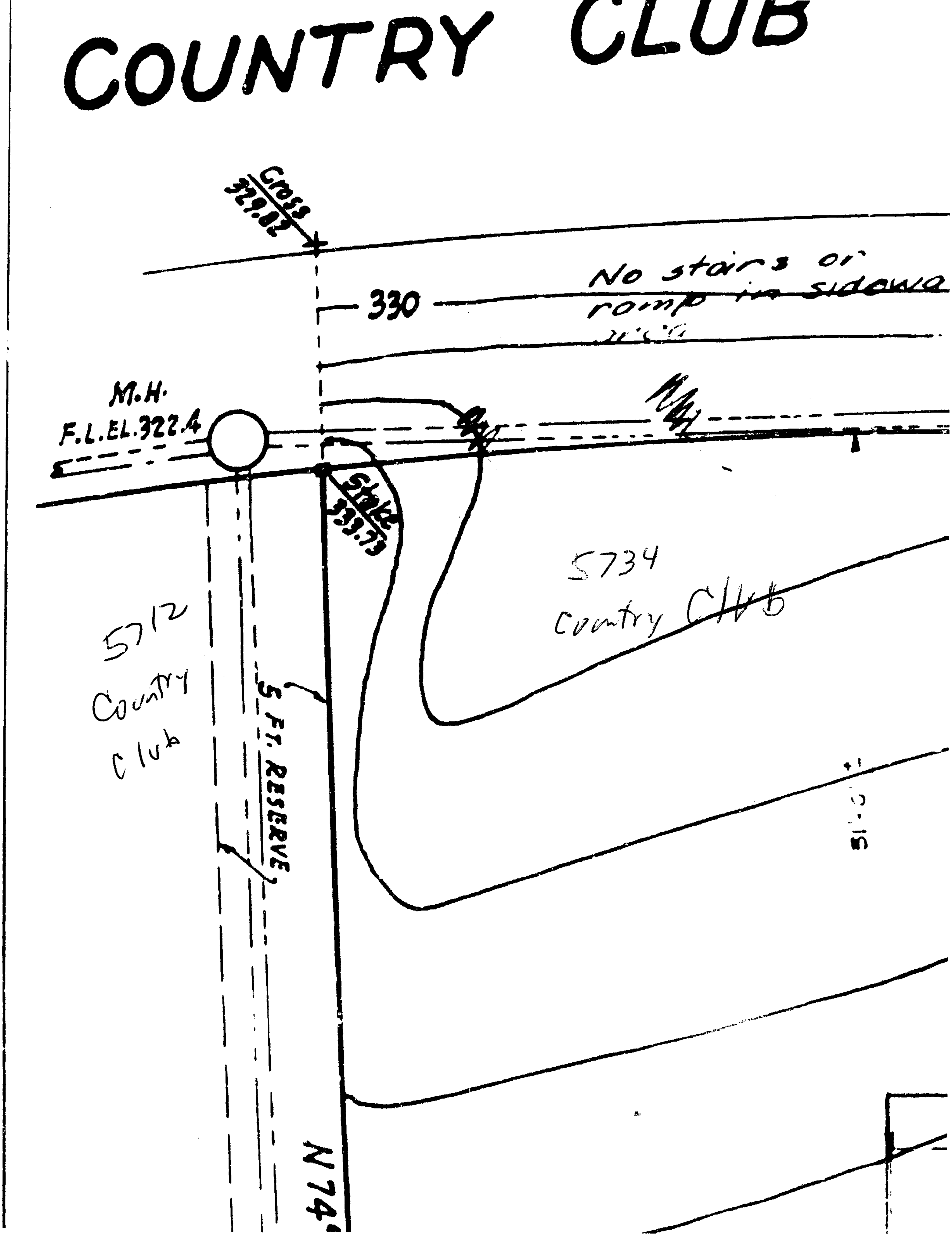
GEOTECNIA Consulting Engineers	Job. No: 020201 Appr: Drwn: CD Date: MAR 2002	DRILLED PIER LATERAL BEHAVIOR Failed Retaining Wall at 5712 Country Club Drive Oakland, California	PLATE 9

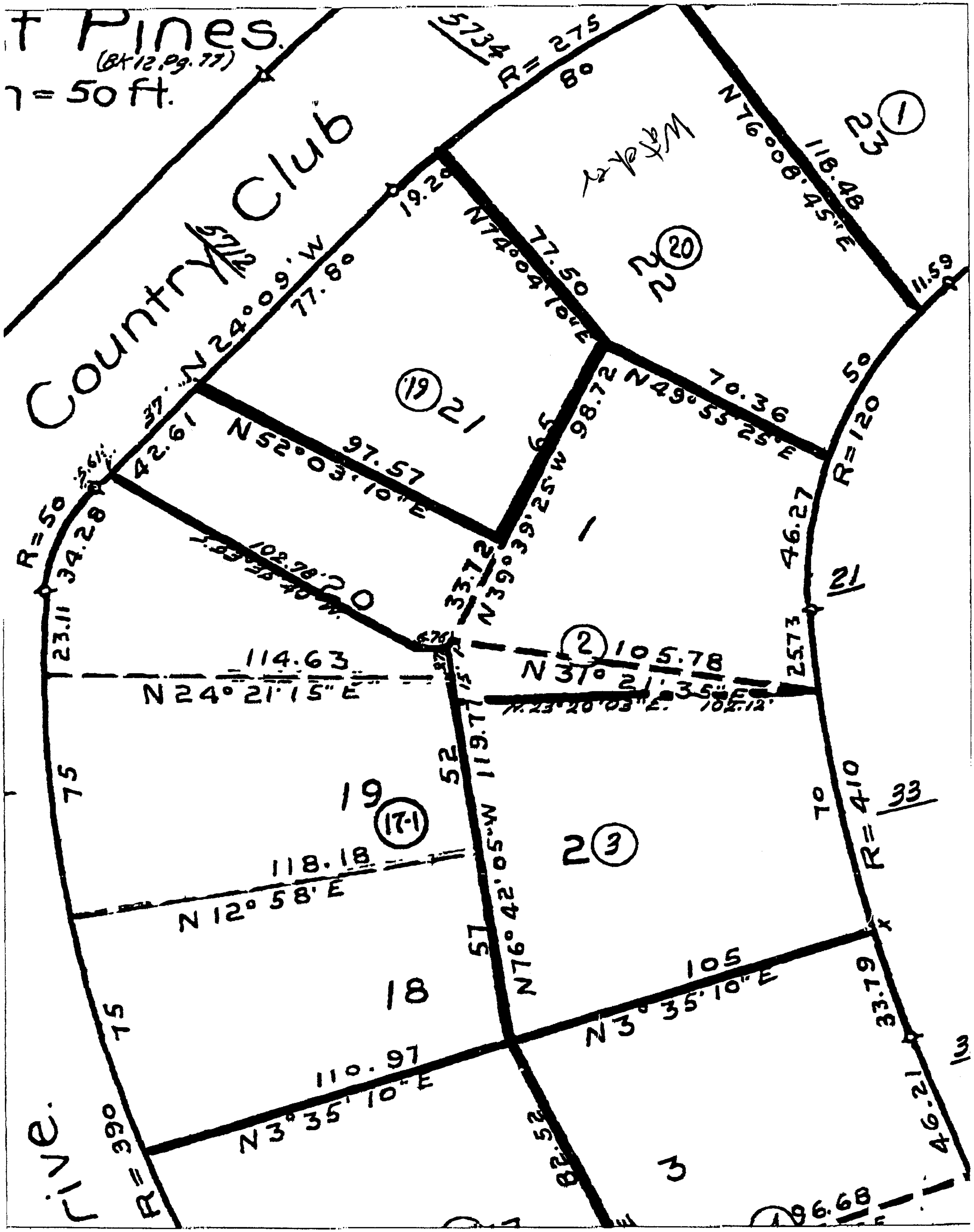


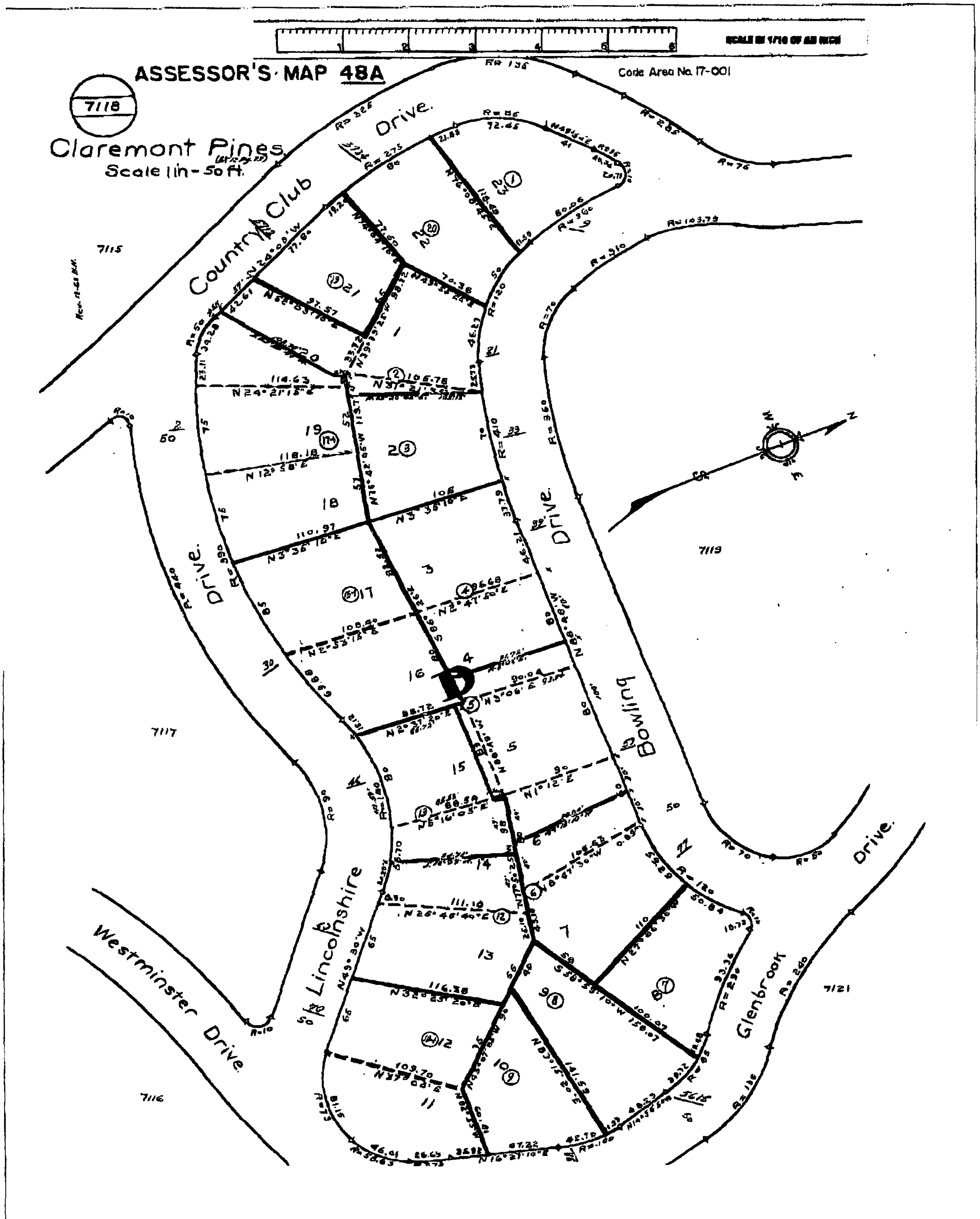
NOTES:

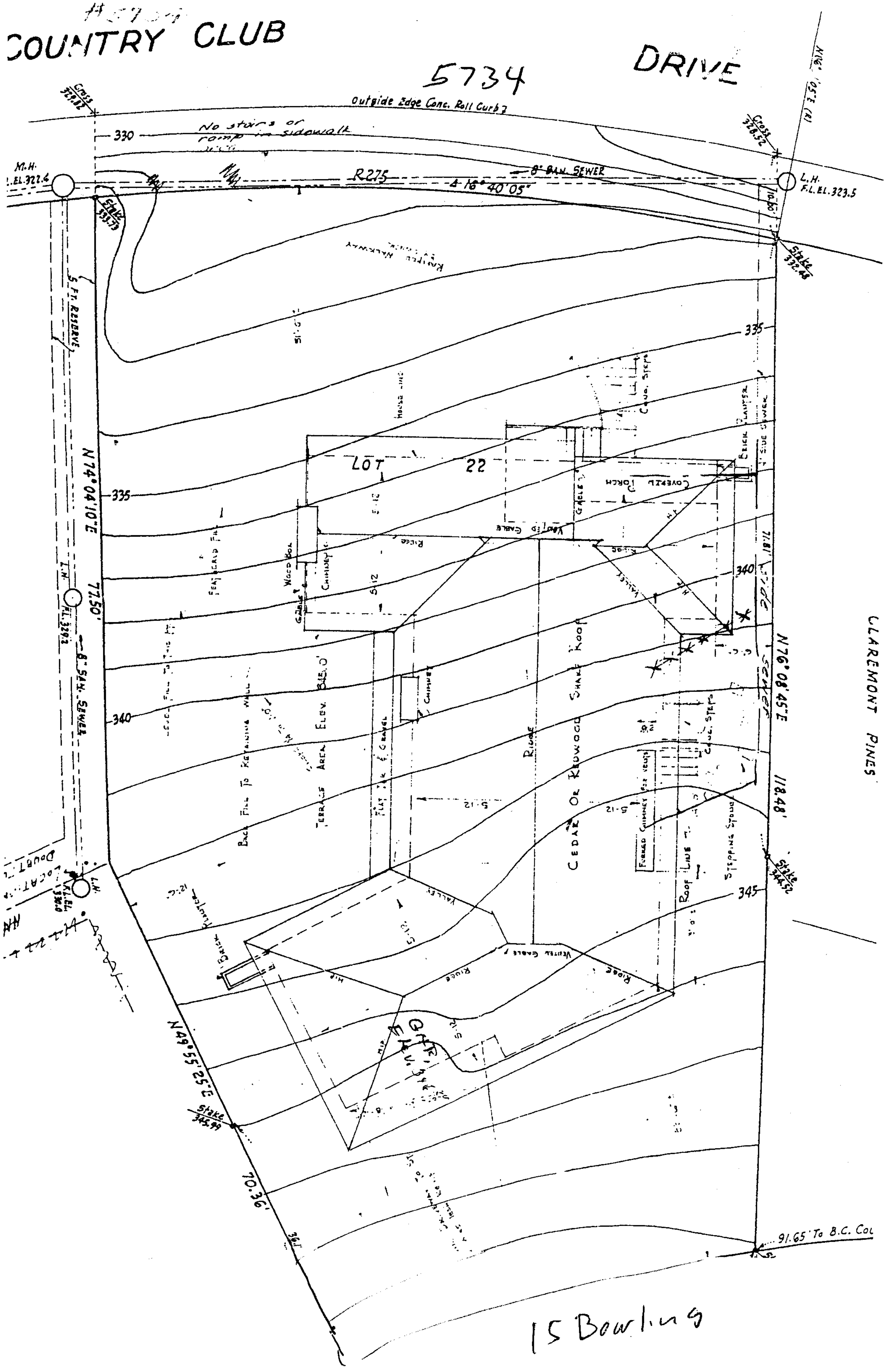
1. For soldier-pier-and-lagging wall, D=0
2. *See text for discussion of values of* γ (equivalent fluid weight)
3. See text for discussion of values of F for seismic surcharge
4. Not to scale

GEOTECNIA Consulting Engineers	Job. No: 020201 Appr: [signature] Drwn: CD Date: MAR 2002	RETAINING WALL LATERAL PRESSURES Failed Retaining Wall at 5712 Country Club Drive Oakland, California	PLATE 10


COUNTRY CLUB
Cross 329.82
330
No stairs or ramp in sidewa
M.H. F.L.EL.322.4
Stake 333.73
5734 Country Club
5712 Country Club
5 FT. RESERVE
N 74°


Pines.
(Bk 12 pg. 77)
= 50 ft.
Country Club
R = 275
80
N76°08'45"E
118.48
N74°04'10"E
77.50
19.20
N24°09'W
77.80
N49°55'25"E
70.36
11.59
50
R=120
N52°03'10"E
97.57
42.61
N39°39'25"W
98.72
65
33.72
46.27
R=50
34.28
23.11
102.78
20
N31°21'35"E
105.78
25.73
114.63
N24°21'15"E
N23°20'03"E
102.12'
119.77
N76°42'05"W
52
57
75
19
2
70
R=410
33
21
118.18
N12°58'E
18
105
N3°35'10"E
33.79
110.97
N3°35'10"E
R=390
82.52
3
46.21
13
96.68


ASSESSOR'S MAP 48A
7118
Claremont Pines
Scale 1in-50ft.
Code Area No. 17-001
Country Club Drive
Bowling Drive
Lincolnshire Drive
Westminster Drive
Glenbrook Drive
7115
7117
7116
7119
7121


COUNTRY CLUB DRIVE
5734
Outside Edge Conc. Roll Curb
No stairs or ramp in sidewalk
R275
8" SAN. SEWER
40°05"
M.H.
L.H.
F.L.EL.323.5
Cross 328.52
Stake 332.48
5 Ft. RESERVE
N74°04'10"E
77.50'
8" SAN. SEWER
LOT 22
COVERED PORCH
VAULTED GABLE
RIDGE
CHIMNEY
WOOD BOX
GABLE
BRICK PLANTER
CONC. STEPS
N76°08'45"E
118.48'
CLAREMONT PINES
BACK FILL TO RETAINING WALL
TERRACE AREA ELEV. 348.0'
FLAT TAR & GRAVEL
CEDAR OR REDWOOD SHAKE ROOF
RIDGE
FORCED CHIMNEY
ROOF LINE
STEPPING STONES
Stake 344.52
335
340
345
VALLEY
HIP
BRICK
N49°55'25"E
70.36'
Stake 345.99
91.65' To B.C. Cou
15 Bowling