Attachment # 7

MR. ALEN PETROSYAN
652 ROBIN GLEN DRIVE
GLENDALE, CA 91202

PRELIMINARY
GEOTECHNICAL ENGINEERING
AND
ENGINEERING GEOLOGY
INVESTIGATION

FOR

652 ROBIN GLEN DRIVE
GLENDALE

Prepared By

SASSAN Geosciences, Inc.
1290 North Lake Avenue, Suite 204
Pasadena, California 91104-2869

January 12, 2018

January 12, 2018

Mr. Alen Petrosyan
652 Robin Glen Drive
Glendale, CA 91202

Subject: Preliminary Geotechnical Engineering
and Engineering Geology Investigation
652 Robin Glen Drive, Glendale, CA 91202
SAS File Number: 7PET140

Dear Mr. Petrosyan:

SASSAN Geosciences, Inc. (SAS) has prepared this preliminary geotechnical engineering and engineering geology investigation for the proposed residence at the subject property, as shown on Figure A-2 in Appendix A of this report.

Our investigation was performed to determine the nature of surface and subsurface soils and to evaluate their physical and engineering properties. The results were then analyzed, and recommendations for foundation design and related parameters were prepared. This report presents our findings and recommendations.

As of the date of this report, no grading or detailed architectural plans have been provided to our office.

LOCATION AND SITE DESCRIPTION

The project site consists of a one-story, single family residence on a graded, flat building pad on the west side of the road at 652 Robin Glen Drive in the City of Glendale. A vicinity map is presented on Figure A-1 in Appendix A of this report.

The home is in steep, hillside terrain that has been graded for development of single-family homes. Existing improvements include an attached garage and large deck and swimming pool in the rear, southern portion of the property. The home is bordered on the east and north by single-family homes, and on the south and west by graded and natural slopes.

In general, the slope gradients in this area are 2:1 (H:V) or gentler. However, locally, the slope gradients may range up to approximately 40 degrees.

OBJECTIVE

The owners wish to assess the geotechnical and geological characteristics of the underlying ground in order to demolish the existing structure, and to construct two-story home with two attached garages, a new swimming pool, and retaining walls on the graded and natural slope area to the west of the residence at the subject property. Implementation of the proposed improvements will require construction of retaining walls up to approximately ten (10) feet in height for the basement, and up to approximately fourteen (14) feet in height for the area to the west of the proposed residence. In addition, the preparation of the building pad and proposed deck areas will require construction and backfill of retaining walls up to approximately eight (8) feet in height. The locations of the existing and the location of the proposed improvements are presented on Figure A-2 in Appendix A of this report.

FIELD INVESTIGATION

Subsurface exploration was performed twice on June 25, 2017 and January 10, 2018 and involved excavating seven (7) test holes and geologic mapping of existing cut slopes for a maximum depth of approximately ten (10) feet. The excavating operation was performed by manual labor. Two and one-half-inch (2.5”) diameter split-spoon ring samples of soil and grab samples of bedrock were obtained from the test holes. Earth materials encountered were classified in accordance with the visual-manual procedures of the Unified Soil Classification System.

A plot plan indicating the approximate test pits locations is presented on Figure A-2 in Appendix A of this report.

GEOLOGIC CONDITIONS

The site is within steep hillside terrain on the southern flank of the Verdugo Mountains, just north of the intersection of the mountainous terrain and the San Fernando Valley south of the site. Published geologic maps (Dibblee, T. W., Jr., 1991, Geologic Map of the Hollywood and Burbank (South ½) Quadrangles) indicate that the site and surrounding terrain are underlain by granitic rocks consisting of quartz diorite. Previous grading during preparation of the area for development of home sites included construction of a steep cut slope on the eastern side of the ridgeline west of the site to provide space for the home and driveway, and grading of a slope to the east that descends to the adjacent home. Descending slopes south and southwest of the site remain in natural condition. A copy of a regional geologic map (Dibblee) is presented on Figure D-1 in Appendix D.

Based on our field investigation, which consisted of excavation of seven test pits and geologic mapping of existing cut slopes, a majority of the building pad and rear deck and pool area are underlain at shallow depth by bedrock exposed during grading of the building pad. The easternmost portion of the building pad and the descending slope on the east side of the property were constructed as compacted fill, as shown on the Geotechnical Plan, Figure A-2, and Cross Sections A-A and B-B, Figures A-3 and A-4 in Appendix A of this report.

Geologic conditions determined by our field investigation and review of published geologic maps are depicted on the Geotechnical Plan, Figure A-2, and Cross Sections, Figures A-3 and A-4 in Appendix A of this report. The locations of exploratory test pits are also indicated on the Plan.

EARTH MATERIALS

The earth materials encountered in the test holes consist of fill up to six and a half (6.5) feet thick, underlain by native soil up to approximately seven and a half (7.5) feet thick, in turn underlain by bedrock which extends to the depths explored. Detailed logs of the test holes are presented on Figures B-1 through B-7 in Appendix B of this report.

GROUNDWATER

Groundwater seepage was not encountered in the test holes to the depths explored, and is not anticipated to impact the proposed construction.

LABORATORY TESTING

Moisture content (ASTM D 2216) and shear strength (ASTM D 3080) tests were performed for selected samples of soil considered to be representative of those encountered. The results of direct shear tests are presented on Figure B-8 in Appendix B of this report. Evaluation of the test data is reflected throughout this report.

LIQUEFACTION

The subject property is shown on the "State of California Seismic Hazard Zones" map presented on Figure C-1 in Appendix C of this report. The site is located outside of the seismically induced liquefaction hazard zones.

CONCLUSIONS AND RECOMMENDATIONS

General

The referenced property is considered to be suitable for construction of the proposed improvements from a geotechnical engineering and engineering geology standpoint, provided that our recommendations are incorporated into the approved construction plans.

The conclusions and recommendations presented here are based on our observations at the site during our investigation, engineering judgment, and analysis of the soil samples obtained from the test holes. Minor variations of subsurface conditions are common, and major variations are possible.

Chain-Link-Fence

We recommend that the owners construct a row of chain-link-fence along the southern and eastern property lines to arrest and retain any potential falling racks and debris. The minimum height of the proposed fence must be six (6) feet. A plot plan and cross-sections indicating the location of the proposed chain-link-fence are presented on Figures A-2, A-3 and A-4 in Appendix A of this report. The contractor may construct the proposed row of chain-link-fence based on the Los Angeles County Road Department Standard Plan 88-01 or similar. A copy of this standard plan is presented in Appendix G of this report.

General Grading

Grading areas must be stripped of all vegetation, debris, and other deleterious material. All loose soil disturbed by the removal of trees and/or structures (if applicable) must be removed and recompacted.

The existing fill and native soil are up to approximately seven and a half (7.5) feet thick and are not suitable for foundation support. At locations where new fill is proposed, the existing fill and native soil must be entirely removed and replaced with a certified engineered fill. The proposed new fill must be placed in horizontal layers, and must be benched into undisturbed bedrock.

Removal of Old Pool

We understand that the owner wishes to remove and backfill the existing old swimming pool and to construct a new swimming pool. The concrete shell of the old swimming pool must be completely removed. Below are provided recommendations for complete removal of the old swimming pool shell:

Initial Preparation – Following initial preparation must be performed prior to backfilling the old swimming pool:

1. Water must be completely drained from the old pool;

2. All organic mater and debris (if any) must be completely removed from bottom of the old pool;

3. All inlet and outlet pipes must be plugged.

Complete Removal of Pool – For complete removal of the old swimming pool, the concrete shell must be cut/broken into small pieces and completely removed from the property. The excavation must be backfilled with engineered fill per recommendations provided later in this report. The new fill must be benched into the sides of the excavation.

Temporary Excavations and Shoring

The review of the architectural plans indicates that excavations up to approximately twelve (12) feet in height may be required during construction of the proposed basement retaining walls, and up to approximately fourteen (14) feet in height for the proposed retaining walls to the west of the proposed residence. The temporary excavations for the proposed basement retaining walls may be performed continuously in accordance with the table provided below. However, due to the topography and heights of the retaining walls to the west of the proposed residence, the construction of these retaining walls must be performed in an "A-B-C" slot-cut manner. The recommendations for slot-cutting are provided.

Based on the integrity of the site earth materials, it is our opinion that unsurcharged temporary excavations may be performed continuously in accordance with the following table:

Maximum Depth of Cut (ft)	Maximum Slope Ratio (H:V)
Bedrock	
0-10	Vertical
>10	1:1
Soil	
0-5	Vertical
>5	1:1

When the above system becomes impractical, shoring has to be designed for the temporary excavations. If such a condition arises, this office can provide the necessary strength parameters needed in the design of shoring elements.

The contractor may perform the excavation under continuous monitoring of a grading inspector who would ensure the quality of grading and presence of competent earth materials. The excavations may be left open for a temporary period of four (4) weeks. A grading inspector must be present when laborers are working within five (5) feet of the temporary cut area.

Slot-Cut Recommendations

The construction of the retaining wall at the toe of the ascending slope in the area to the west of the proposed residence must be performed in an “A-B-C” slot-cut manner. The following are our recommendations for slot-cutting:

1. The entire length of the proposed excavation must be divided into 8-foot long equally spaced segments. The results of the analysis for stability of the temporary excavations are presented in Appendix E of this report.

2. The segments must be designated “A”, “B”, “C”, “A”, “B”, “C” and so on.

3. Only “A” segments may be excavated at the same time.

4. Bottom preparation of the slot-cut excavations must be inspected and approved by the consulting soils engineer.

5. Place the reinforcing steel of the foundation per approved plans, and place additional horizontal rebar extensions in the excavations. The extensions must be bent at the ends of the segments. These extensions are to be straightened during rebar placement in the adjacent segments.

6. Pour the retaining wall footing with concrete and construct the wall.

7. After the concrete obtains the required strength, install a subdrain system and place engineered backfill behind the retaining wall.

8. Excavate segments “B” and repeat steps 4 through 7.

9. Excavate segments “C” and repeat steps 4 through 7.

10. A soils inspector approved by and responsible to this office will be required to provide continuous inspection during the proposed slot cutting.

Foundations

The foundations of the proposed improvements, including residence, garages, swimming pool and retaining walls must derive their support from undisturbed bedrock. The parts of the proposed residence and retaining walls within the western and northern portions of the building pad, that have the minimum Code required daylight distance from the descending slopes, may be supported by conventional continuous and spread footings. However, the parts of the proposed residence, swimming pool and retaining walls within the eastern and southern portions of the proposed residence, that are located closer to the descending slopes, must be supported by grade beam/friction pile combination foundation.

Following are our recommendations for design of the proposed structures and both types of foundations:

Conventional Foundations - The parts of the proposed residence and retaining walls within the western and northern portions of the building pad, that have the minimum Code required daylight distance from the descending slopes, may be supported by conventional continuous and spread footings. The footings must be founded into undisturbed bedrock. In addition, the bottoms of proposed footings must be below a plane with a slope of one horizontal to one vertical (1:1) projected upward from the bottom edge of adjacent existing footings (if any).

An allowable bearing capacity of up to the maximum value of 3,000 psf may be used for footings eighteen (18) inches wide and founded eighteen (18) inches into undisturbed bedrock. The allowable bearing capacity may be increased by twenty (20) percent for every additional foot of width or depth to a maximum value of 4,500 psf.

The allowable bearing value is for dead-plus-live loads and may be increased by thirty (30) percent for momentary wind and seismic loads. The following minimums apply to all footings:

1. Footings must be founded at a minimum depth of eighteen (18) inches into undisturbed bedrock.

2. Footings must be reinforced with a minimum of four (4) #4 bars - two at the top and two at the bottom. The final design of the footings must be provided by a structural engineer in conjunction with this office.

3. An at-rest earth pressure increasing at a minimum rate of 62 psf per foot of depth must be used in the design of the proposed basement retaining walls that are braced at the top and the bottom. Our analyses indicate, that the calculated pseudo-static earth pressure (from combined static and seismic forces) is below the above recommended at-rest earth pressure, therefore additional earth pressure due to seismic forces does not need to be applied to the retaining walls of the proposed basement. The results of the active pressure analysis are presented in Appendix F of this report.

4. Active earth pressure increasing at rates listed in the following table must be used in the design of the retaining wall at the toe of the slope to the west of the proposed residence. Our analyses indicate, that the calculated pseudo-static earth pressure (from combined static and seismic forces) is less than the below recommended active earth pressures, therefore additional earth pressure due to seismic forces does not need to be applied to the retaining wall to the west of the proposed residence. The results of the active pressure analysis are presented in Appendix F of this report.

Slope of the Retained Backfill (H:V)	*Active EFP (pcf)*
Level	30
5:1	33
4:1	35
3:1	38
2:1	43
1½:1	55
1:1	80

5. Passive earth pressure increasing at a maximum rate of 400 psf per foot of depth, to a maximum of 4,000 psf, must be used in calculations for portions of the footings that are in contact with bedrock.

6. A coefficient of friction of 0.4 must be utilized for resisting lateral loads at the contact surface of concrete and foundation soils.

7. Frictional and passive resistance of end-bearing foundations may be combined, provided the passive bearing resistance does not exceed two-thirds of the allowable passive bearing.

8. A minimum setback distance of fifteen (15) feet must be considered for the proposed residence from the toe of the westerly ascending slope. The recommended setback distance must be measured horizontally from the toe-of-slope retaining wall face in the direction perpendicular to the slope contours.

9. A minimum daylight distance of eight (8) feet must be considered for all footings on or near easterly descending slope, and a minimum daylight distance of forty (40) feet must be considered for all footings on or near southerly descending slope. The recommended daylight distance must be measured horizontally from the soil/bedrock contact.

Friction Piles - The parts of the proposed residence, swimming pool and retaining walls within the eastern and southern portions of the proposed residence, that are located closer to the descending slopes, must be supported by grade beam/friction pile combination foundation. The following recommendations should be implemented. An allowable side friction value of 500 psf in compression and 250 psf in tension may be utilized for the portion of the friction piles that are penetrated into bedrock. The allowable side friction values may be increased by thirty (30) percent for momentary wind and seismic loads. The following minimums apply to the friction piles:

1. Friction piles must be founded at a minimum depth of eight (8) feet into undisturbed bedrock. The actual depth of friction piles, however, must be determined by the structural engineer in conjunction with this office.

2. Friction piles must have a minimum diameter of twenty-four (24) inches.

3. The pile excavations must be covered if left overnight.

4. A soils inspector approved by and responsible to this office will be required to provide continuous inspection for the proposed friction pile drilling and installation.

5. Active earth pressure increasing at a minimum rate of 33 psf per foot of depth must be used in the design of cantilevered retaining walls required for preparation of the building pad and proposed deck areas to the east and south of the proposed residence. Our analyses indicate, that an additional earth pressure due to seismic forces increasing at a minimum rate of 9 psf per foot of depth must be applied to these retaining walls in a form of an inverted triangle, and the resultant force due to seismic pressure must be applied at an elevation equal to 60 per cent of the retained height. The results of the active pressure analysis are presented in Appendix F of this report.

6. A minimum creep load of 1,000 plf must be applied to the portions of the friction piles that are in contact with the fill and residual soil.

7. Passive earth pressure increasing at a maximum rate of 400 psf per foot of depth, to a maximum of 4,000 psf, must be applied to portions of the friction piles that are embedded a minimum two (2) feet into undisturbed bedrock.

8. The suggested passive pressure may be doubled for an isolated pile condition (d>2.5D).

9. A minimum daylight distance of eight (8) feet must be considered for all foundations on or near the easterly descending slope, and a minimum daylight distance of forty (40) feet must be considered for all foundations on or near the southerly descending slope. A minimum daylight distance of twenty (20) feet must be considered for the proposed swimming pool foundations on or near the southerly descending slope. The daylight distance must be measured horizontally from the soil/bedrock contact. The recommended daylight distance must be measured horizontally from the soil/bedrock contact.

10. Caving may be encountered within the fill and residual soil during the drilling of the friction piles. As such, the contractor is advised to have steel casing ready to use if caving occurs.

Subdrain System

The retaining walls must be provided with weep holes or perforated pipe and gravel sub-drain to prevent entrapment of water in the backfill. The perforated pipe must consist of four-inch (4”) minimum diameter PVC Schedule 40, or ABS SDR-35, with a minimum of sixteen (16) perforations per foot on the bottom one-third of the pipe. Every foot of the pipe should be embedded in three (3) cubic feet of three-quarter-inch (3/4”) gravel wrapped in filter fabric (Mirafi 140N or equal). Placement of gravel and filter fabric is also required for weep holes.

In addition, the retaining walls of the basement must be provided with extensive damp-proofing. The damp-proofing must be designed by a water proofing specialist.

Swimming Pool Recommendations

The concrete shell of the proposed swimming pool must be designed as a freestanding structure. The swimming pool shell must be supported by grade beam/friction pile combination foundation per recommendations provided in the previous section of this report. The following minimums apply to the lateral loading of the concrete shell of the proposed swimming pool:

1. Water pressure increasing at the rate of 65 psf per foot of depth and acting upon the inner wall of the shell must be used in calculations.

2. Active earth pressure increasing at a minimum rate of 65 psf per foot of depth and acting upon the outer wall of the shell must be used in calculations.

Engineered Fill

All fill earth materials must consist of clean soil that is free of vegetation and other debris. The fill must be placed in six- (6-) to eight- (8-) inch thick lifts at near optimum moisture content and compacted. Particles larger than three (3) inches in diameter must not be allowed in the backfill material. Earth materials must not be imported to the site without prior approval by the soil engineer. All engineered fill must be compacted to a minimum of ninety (90) percent of its maximum dry density (ASTM D 1557). Where cohesionless soil having less than fifteen (15) percent finer than 0.005 millimeter is used for fill, it must be compacted to a minimum of ninety-five (95) percent of its maximum dry density. Neither jetting nor water tamping are permitted.

Heavy construction equipment must be maintained at a minimum distance of three (3) feet from the existing structures. Hand-operated compaction equipment must be used to compact the backfill soils within this 3-foot-wide zone.

Settlements

Maximum total and differential settlements are expected to be less than one-half (½) and one-quarter (¼) inches, respectively, provided that our recommendations are followed.

Seismic Hazards

The subject property is shown on the "State of California Seismic Hazard Zones" map presented in Appendix C of this report. The site is located outside of potential seismically induced landslide and liquefaction hazard zones.

Seismic Parameters

The seismic parameters for the design of the proposed structure based on the most recent California Building Code are as follows:

Latitude	34^{o} 10' 37" N
Longitude	118^{o} 16' 12" W
Site Classification	C
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.3
Site Spectral Response Acceleration Parameters (g):	
Mapped Acceleration, S_S (0.2 sec.)	2.802
Mapped Acceleration, S_1 (1 sec.)	0.970
Adjusted Maximum Acceleration, S_{MS} (0.2 sec.)	2.802
Adjusted Maximum Acceleration, S_{M1} (1 sec.)	1.261
Design Acceleration, S_{DS} (0.2 sec.)	1.868
Design Acceleration, S_{D1} (1 sec.)	0.840

Conformance with the above listed criteria for seismic design does not constitute any kind of warranty, guarantee, or assurance that significant structural damage or ground failure will not occur if a maximum level earthquake occurs. The primary goal of seismic design is to protect life and limb, and to prevent catastrophic failures, and not to avoid all damage, since such design may be economically prohibitive.

Internal Concrete Slabs

The subgrade for the proposed concrete slabs-on-grade must consist of a minimum two (2) foot thick layer of certified compacted fill. The competent subgrade must be covered with four (4) inches of crushed miscellaneous aggregate (CMA) and compacted to ninety-five percent (95%) of its maximum dry density (ASTM D 1557). The CMA must be covered with one (1) inch of sand. The sand must be covered by a ten (10)-mil vapor barrier. The vapor barrier must be installed so that the edges of the sheet overlap at least twelve (12) inches onto any adjacent sheet. The vapor barrier must be covered with one (1) inch of sand. The sand must be covered with four (4) inches of non-expansive hard rock concrete mix (3/4" max. rock size). The reinforcement must be a minimum of #4 bars at sixteen (16) inches on center in both directions. The reinforcement must be placed at the mid-depth of the concrete slab. The slab must be covered with a vapor barrier for at least two (2) days to slow the curing time, reduce the shrinkage crack potential and be self-watering.

The consulting structural-engineer-of-record may decide to increase the slab thickness according to the proposed traffic loads.

Pipe Bedding and Trench Backfill

The pipe bedding must consist of sand or similar granular material having a minimum sand equivalent value of thirty (30). The sand must be placed in a zone that extends a minimum of six (6) inches below and twelve (12) inches above the pipe for the full trench width. The bedding material must be compacted. The trench backfill above the pipe bedding may consist of approved, on-site or imported soils, and it must be compacted. Where utility trenches are parallel to the footings, the bottom of the trench must be located above a plane with a slope of 1:1, projected downward from the adjacent bottom edge of the footing.

Site Drainage

Drainage devices such as sloping sidewalks and area drains must be provided around the building to collect and direct all water away from the structure. Neither rain nor excess irrigation water should be allowed to collect or pond against foundations. The collected water must be directed to the proper drainage system via non-erosive devices. The actual site drainage, however, must be designed by the consulting civil engineer-of-record.

DESIGN REVIEW

We suggest that the geotechnical and geological aspects of the project be reviewed by this firm during the design process. The scope of our services may include assistance to the design team by providing specific recommendations for special cases, reviewing the foundation design, reviewing the geotechnical and geological portions of the project for possible cost savings through alternative approaches, and evaluating the overall applicability of our recommendations. Additional site-specific explorations may also be considered if significant foundation modifications are required using the above recommendations.

The owner should anticipate that both the geologist and soils engineer must review and approve the detailed plans prior to issuance of any permits. This approval shall be by signature on the plans which clearly indicates that the geologist and soils engineer have reviewed the plans prepared by the design engineer and that the plans include the recommendations contained in their reports.

INSPECTION

All excavations must be inspected and approved. All fill placed for engineering purposes must be tested for compaction and moisture content and certified. Inspection of excavations may also be required by the appropriate reviewing governmental agencies.

It is recommended that SAS be retained to verify compliance with the recommendations made in this report, to ensure compliance with the design concepts, specifications, and recommendations, and to allow design changes in the event that exposed subsurface conditions differ from those anticipated herein.

A joint meeting among the parties involved in this project is recommended prior to the start of groundbreaking to discuss specific procedures and scheduling.

Inspections performed by SAS are for verification purposes only and shall under no circumstance relieve other parties involved in the design and construction from their obligation to perform work in accordance with the approved plans.

In the event that the recommendations contained herein are interpreted by others, SAS will not accept responsibility for such interpretations.

INVESTIGATION LIMITATIONS

The conclusions and recommendations presented in this report are based on the findings and observations in the field and the results of laboratory tests performed on representative samples. The soils encountered in the test holes are believed to be representative of the total area; however, soil characteristics can vary throughout the site. SAS should be notified if subsurface conditions are encountered which differ from those described in this report.

This report has not been prepared for use by parties or projects other than those named and described above. It may not contain sufficient information for other parties or other purposes. The conclusions and recommendations presented in this report are professional opinions. These opinions have been derived in accordance with current standards of geotechnical engineering and engineering geology practice, field observations and laboratory test results. No other warranty is expressed or implied.

Samples secured for this investigation will be retained in our laboratory for a period of thirty (30) days from the date of this report and will be disposed after this period unless other arrangements are made.

This report should be reviewed and updated after a period of one year or if the project concept changes from that described herein.

We appreciate the opportunity to be of service to you. If you have any questions, please call our office.

Sincerely,

SASSAN GEOSCIENCES, INC.

REGISTERED PROFESSIONAL ENGINEER
SASSAN A. SALEHIPOUR
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Janan Anayi, Ph.D.
Project Manager

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Appendices

REFERENCES:

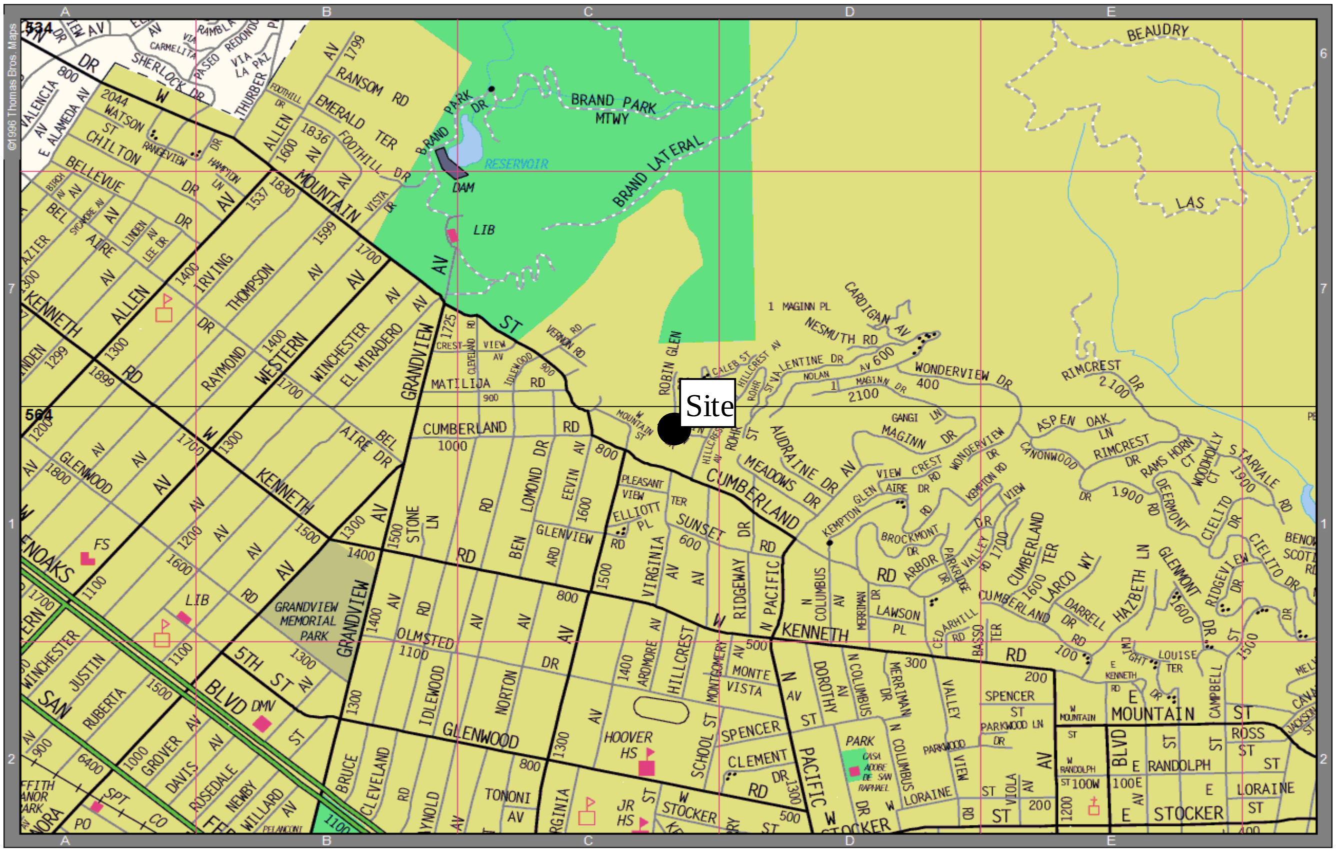
1. California Division of Mines and Geology, 1998, Seismic Hazard Evaluation for the Burbank 7.5 Minute Quadrangle, Los Angeles County, California; OFR 98-07

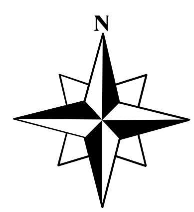
2. California Division of Mines and Geology, 1999, Seismic Hazard Zones Map for the Burbank Quadrangle, Los Angeles County, California; Scale 1:24,000

3. California Division of Mines and Geology, 1979, Official Map of Special Studies Zones, Burbank Quadrangle; Scale 1:24,000

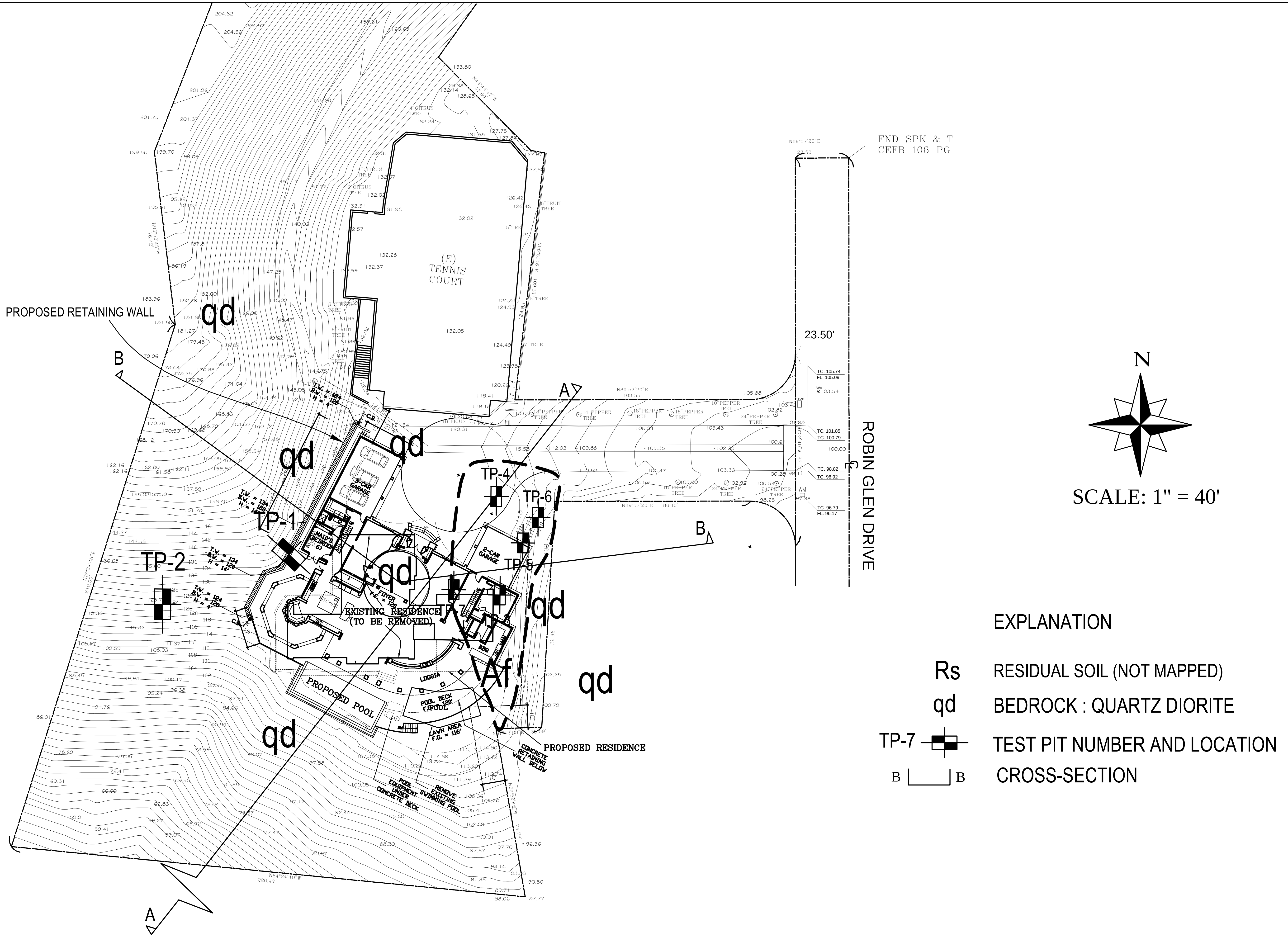
4. Dibblee, T.W., 1991, Geologic Map of the Hollywood and Burbank (South ½) Quadrangles, Los Angeles County, California. Dibblee Geological Foundation , Santa Barbara, California; Map DF-30; Scale 1:24,000

APPENDIX A





SAS	VICINITY MAP 652 ROBIN GLEN DRIVE, GLENDALE	FIGURE A-1



EXPLANATION

Rs	RESIDUAL SOIL (NOT MAPPED)
qd	BEDROCK : QUARTZ DIORITE
TP-7	TEST PIT NUMBER AND LOCATION
B B	CROSS-SECTION

FIGURE A-2

PLOT PLAN

652 ROBIN GLEN DRIVE, GLENDALE

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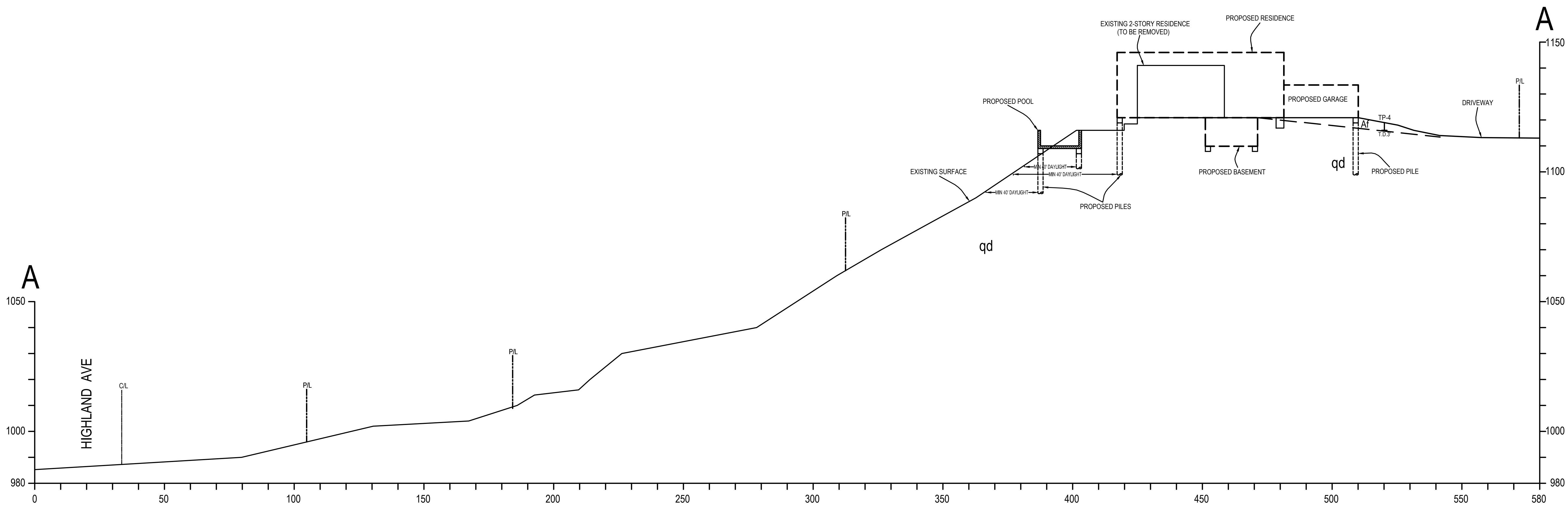

SCALE : 1" = 40'

FIGURE
A-3

CROSS-SECTION A-A
652 ROBIN GLEN DRIVE, GLENDALE

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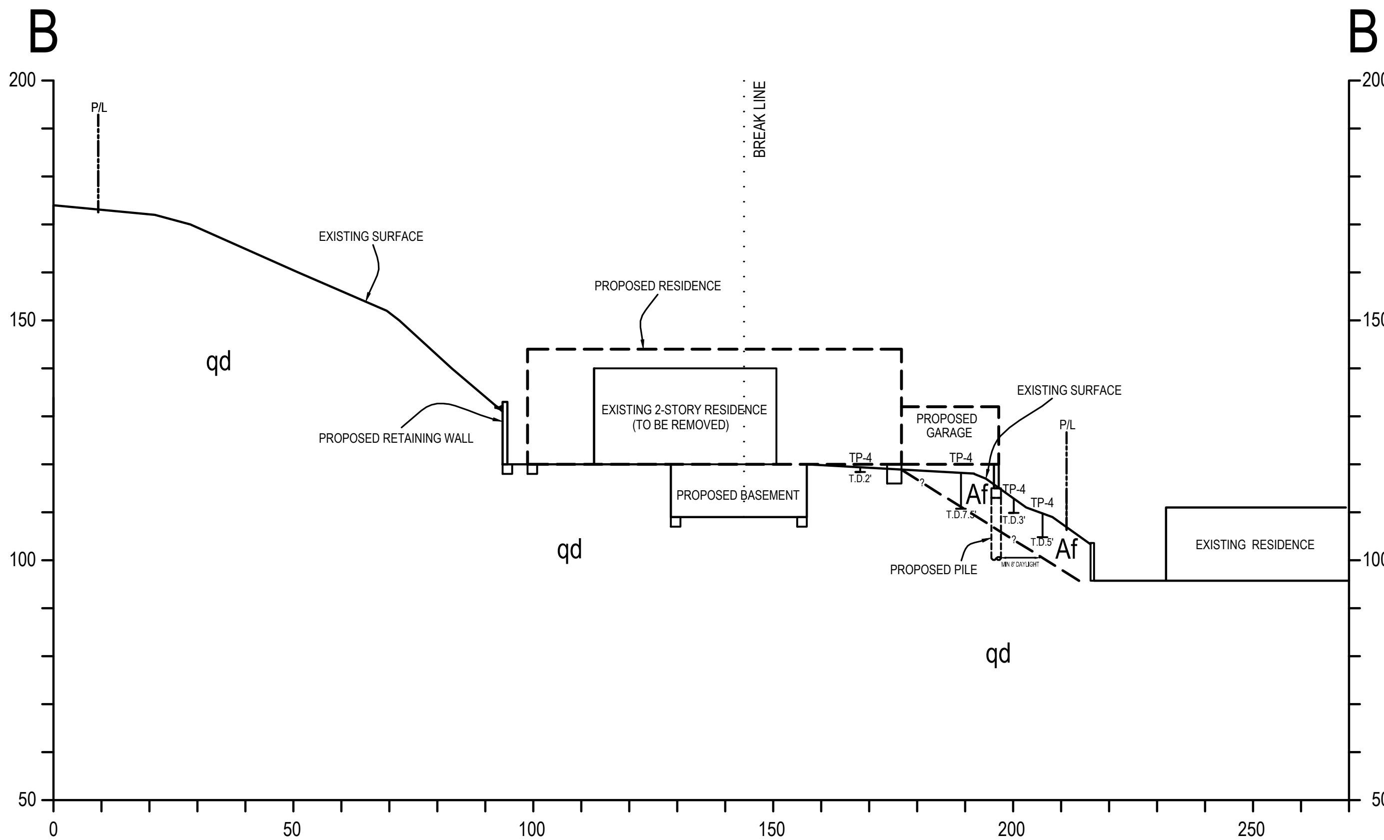
CROSS-SECTION B-B
652 ROBIN GLEN DRIVE, GLENDALE



SCALE : 1" = 40'

FIGURE
A-4

APPENDIX B

Sample Number	γ_d (pcf)	Moisture (%)	N	USCS	Depth (ft)	Description
G-1	134	4		BX	0–10	**Exposed Cut (Bedrock): Quartz Diorite** Brown, moderately medium crystalline, quartz diorite. Hard difficult to excavate
G-2	138	5				
G-3	136	4				
					10–20	Excavation Terminated at Depth of 10 Feet Water Seepage Was Not Encountered

G = Grab Sample

SAS	LOG OF TEST PIT NUMBER ONE (TP-1) 652 ROBIN GLEN DRIVE, GLENDALE	FIGURE B-1

Sample Number	γ_d (pcf)	Moisture (%)	N	USCS	Depth (ft)	Description
				GP	0	**Native (Residual Soil):** Brown silty gravel of angular diorite fragments. Slightly moist, loose, many rootlets
G-1	136	4		BX	2	**Bedrock:** Mottled gray and brown, medium crystalline quartz diorite moderate to highly fractured. Hard. Difficult to excavate.
					4	
					6	
					8	
					10	
					12	
					14	
					16	
					18	
					20	Excavation Terminated at Depth of 2 Feet Water Seepage Was Not Encountered

G = Grab Sample

SAS	LOG OF TEST PIT NUMBER TWO (TP-2) 652 ROBIN GLEN DRIVE, GLENDALE	FIGURE B-2

Sample Number	γ_d (pcf)	Moisture (%)	N	USCS	Depth (ft)	Description
R-1	111	6		SM	0–7.3	**Fill:** Light brown, silty, fine to coarse sand with gravel of granitic rock fragments. Moist at surface (to 12") then slightly moist. Loose to medium dense
R-2	120	7		SM		**Native:** Brown silty sand with granitic rock fragments. Porous with root mold, decayed roots. Slightly moist. Refusal on rocks

Depth scale (ft): 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20

Excavation Terminated at Depth of 7.3 Feet
Water Seepage Was Not Encountered

R = Ring Sample

Sample Number	γ_d (pcf)	Moisture (%)	N	USCS	Depth (ft)	Description
R-1	114	6		SP	0 2	**Fill:** Light brown, silty, fine to coarse sand with granitic rock fragments to 4"-6". Slightly moist. Dense, difficult to excavate at 3 ft. Few fill 4'-6' thick
					4 6 8 10 12 14 16 18 20	Excavation Terminated at Depth of 3.0 Feet Water Seepage Was Not Encountered

R = Ring Sample

SAS	LOG OF TEST PIT NUMBER FOUR (TP-4) 652 ROBIN GLEN DRIVE, GLENDALE	FIGURE B-4

Sample Number	γ_d (pcf)	Moisture (%)	N	USCS	Depth (ft)	Description
R-1	117	5		SP	0–3	**Fill:** Light brown, silty, fine to coarse sand with granitic rock fragments to 4". Slightly moist. Dense, massive
					0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20	

Excavation Terminated at Depth of 3.0 Feet
Water Seepage Was Not Encountered

R = Ring Sample

SAS	LOG OF TEST PIT NUMBER FIVE (TP-5) 652 ROBIN GLEN DRIVE, GLENDALE	FIGURE B-5

Sample Number	γ_d (pcf)	Moisture (%)	N	USCS	Depth (ft)	Description
R-1	121	6		SP	0–5	**Fill:** Light brown, silty, fine to coarse sand with granitic rock fragments to 4" to 6". Slightly moist. Dense
					5–20	

Excavation Terminated at Depth of 5.0 Feet
Water Seepage Was Not Encountered

R = Ring Sample

SAS	LOG OF TEST PIT NUMBER SIX (TP-6) 652 ROBIN GLEN DRIVE, GLENDALE	FIGURE B-6

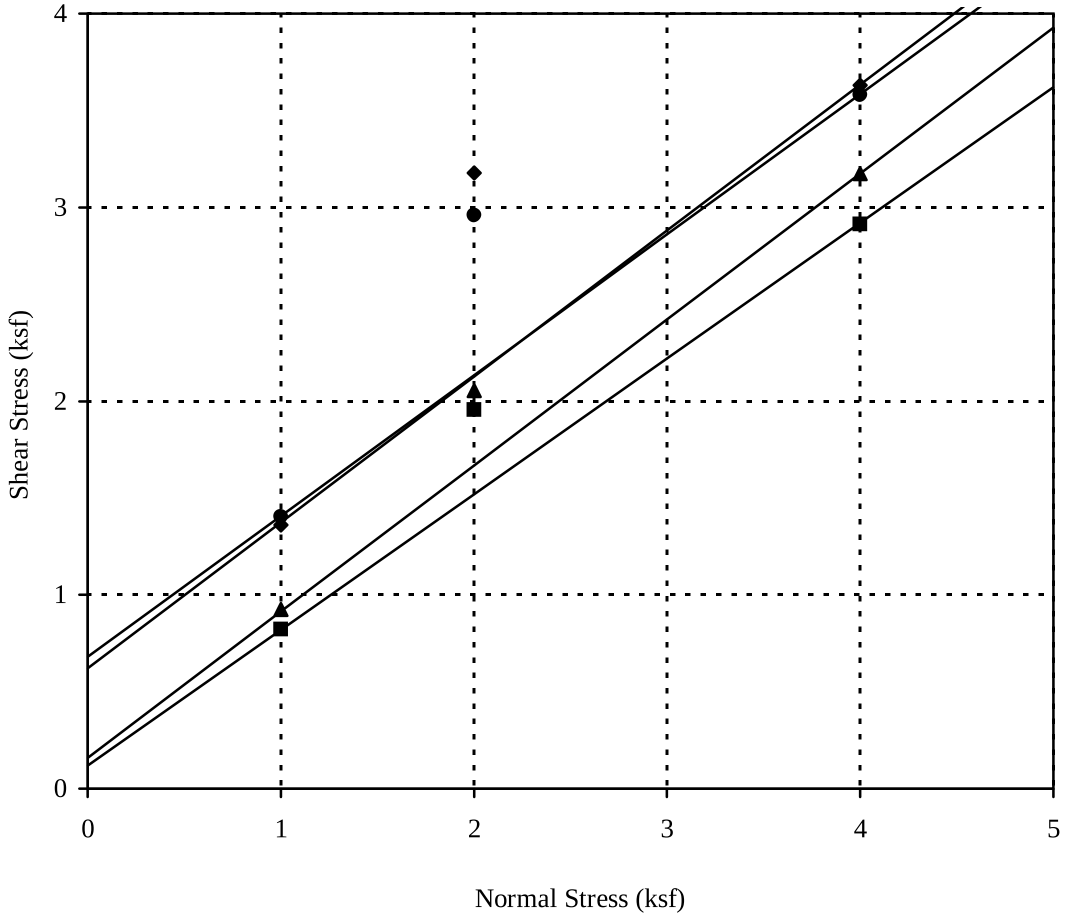
Sample Number	γ_d (pcf)	Moisture (%)	N	USCS	Depth (ft)	Description
R-1	116	5		SP	0–2	**Fill:** Light brown, silty, fine to coarse sand with rock fragments. Very dense, difficult to excvate. Slightly moist
					2–20	

Depth scale: 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20

Excavation Terminated at Depth of 2.0 Feet
Water Seepage Was Not Encountered

R = Ring Sample

SAS	LOG OF TEST PIT NUMBER SEVEN (TP-7) 652 ROBIN GLEN DRIVE, GLENDALE	FIGURE B-7



Symbol	Test Location	Sample Number	Depth (ft)	Soil Type	Cohesion (psf)	Friction Angle (deg)	Remarks
u	TP-1	G-2	4	Bedrock	620	37	1
●	TP-2	G-1	2	Bedrock	680	36	2
■	TP-3	R-2	7	SM	120	35	3
▲	TP-6	R-1	3	SM	160	37	4

Remarks:

1 - BEDROCK; Saturated Moisture Content: 8.5%, Dry Density: 135 pcf; Ultimate

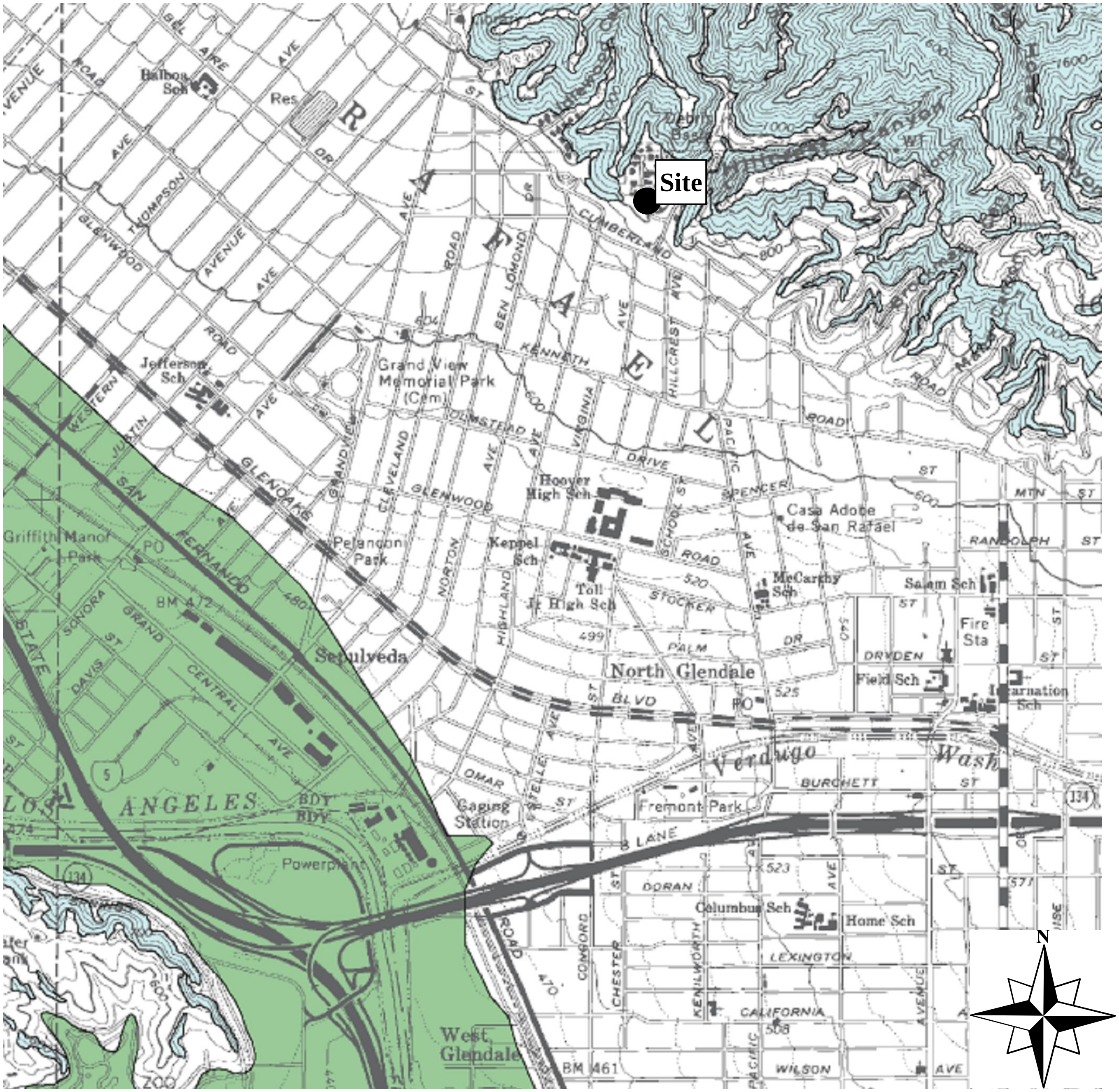
2 - BEDROCK; Saturated Moisture Content: 8%, Dry Density: 136 pcf; Ultimate

3 - NATIVE; Saturated Moisture Content: 14%, Dry Density: 120 pcf; Ultimate

4 - FILL; Saturated Moisture Content: 13.5%, Dry Density: 121 pcf; Ultimate

SAS	DIRECT SHEAR TEST RESULTS 652 ROBIN GLEN DRIVE, GLENDALE	FIGURE B-8

APPENDIX C



STATE OF CALIFORNIA

SEISMIC HAZARD ZONES

Delineated in compliance with
Chapter 7.8, Division 2 of the California Public Resources Code
(Seismic Hazards Mapping Act)

BURBANK QUADRANGLE

OFFICIAL MAP

Released: March 25, 1999

MAP EXPLANATION

Zones of Required Investigation:

Liquefaction

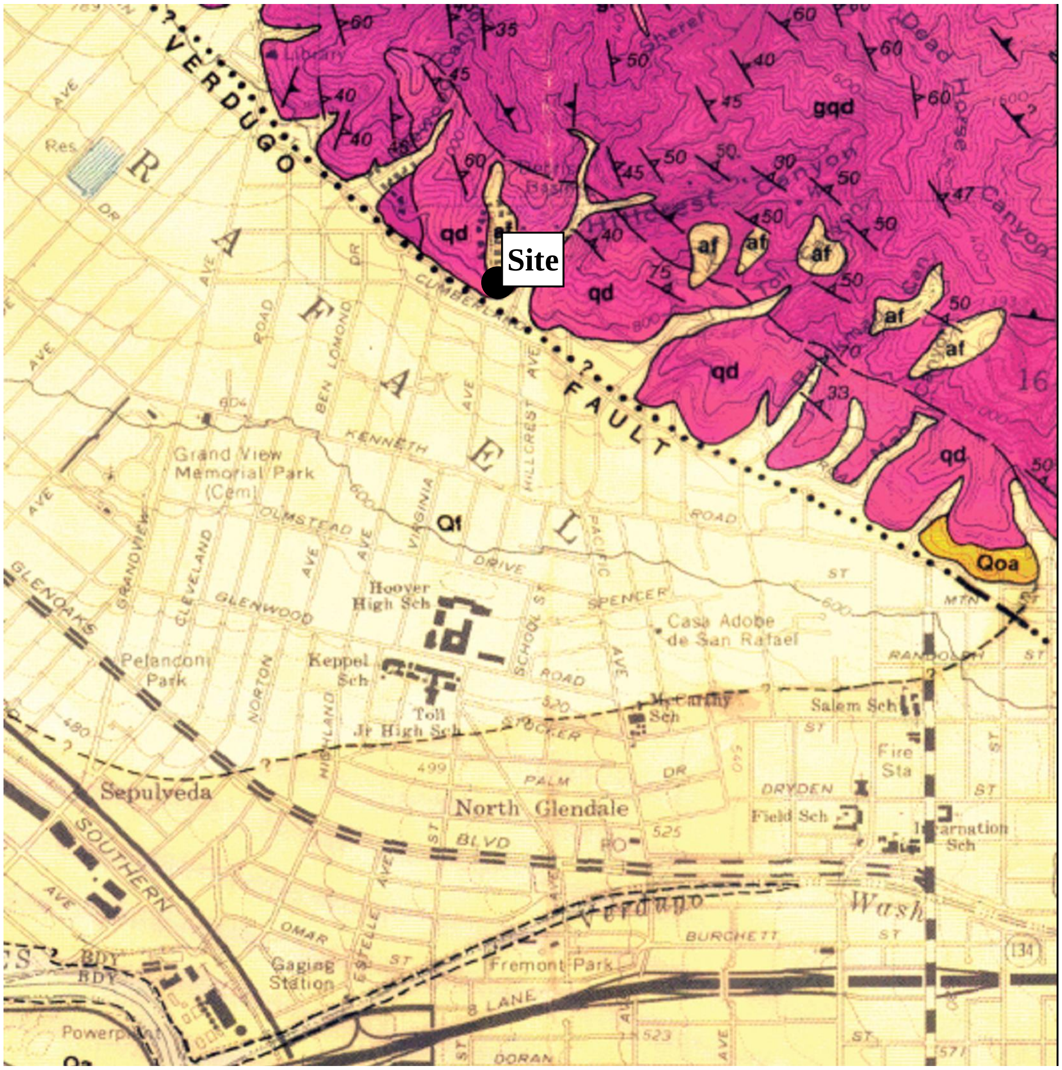
Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

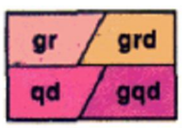
Earthquake-Induced Landslides

Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

SAS	CALIFORNIA SEISMIC HAZARD ZONES MAP 652 ROBIN GLEN DRIVE, GLENDALE	FIGURE C-1

APPENDIX D





GRANITIC ROCKS

late Mesozoic (Cretaceous) age

medium-grained granitic rocks composed mostly of plagioclase feldspar (oligoclase-andesine) and lesser amounts of potash feldspar (microcline), quartz, biotite, and hornblende

gr *nearly white, massive, medium to fine grained granitic rocks mostly of quartz monzonite-granodiorite composition; essentially of quartz, potassic feldspar and sodic plagioclase feldspar, sparse biotite; complexly intruded into* **qd** *and* **gqd** *in Verdugo Mountains; moderately coherent*

grd *granodiorite (Feliz biotite granodiorite of Neuerburg 1953), light gray, massive moderately hard; composed mostly of plagioclase feldspar, lesser amounts of quartz, potassic feldspar and biotite; intrusive(?) into* **qd** *(Neuerburg 1953); moderately coherent*

qd *quartz diorite (Lar and Vermont biotite quartz diorite of Neuerburg 1953, in Griffith Park area), medium to light gray, massive to vaguely gneissoid; composed mostly of plagioclase feldspar, and moderate amounts of quartz, biotite, and hornblende; moderately hard to somewhat incoherent where weathered*

gqd *gneissoid quartz diorite, similar to* **qd**, *but gneissoid; in north Glendale area somewhat porphyritic with small phenocrysts of white feldspar and of black hornblende in dark, very fine grained groundmass; in many places contains remnants of gneiss, but too small to map; locally contains dark gray-brown andesite dikes also too small to map*

N

Dibblee, T.W., Jr., Dibblee Geological Foundation, Geologic Map of Hollywood and Burbank (South ½) Quadrangles, 1991, Map # DF-30

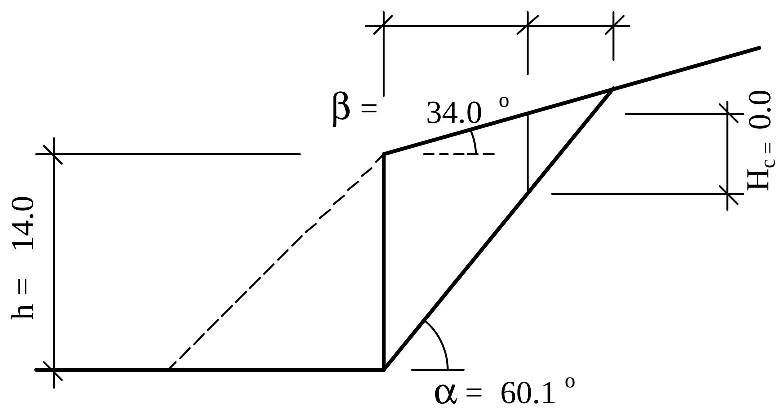
SAS	GEOLOGIC MAP 652 ROBIN GLEN DRIVE, GLENDALE	FIGURE D-1

APPENDIX E

WIDTH OF THE SLOT CUT
FOR 14 FEET HIGH EXCAVATION
(BEDROCK)

Data:

Height of Cut,	$h =$ 14.0	ft
Slope Angle,	$\beta =$ 34.0	deg
Density of Soil,	$\gamma_s =$ 147	pcf
Cohesion,	$C =$ 680	psf
Friction Angle,	$\phi =$ 36	deg
Factor of Safety,	F.S. = 1.25	



Maximum Width of Slot:

$$d = \frac{1/3 * \gamma_s * K_o * \tan\phi * (h^2 * (a + b) - H_c^2 * a) + 2A*C}{(F.S.) * W * \sin\alpha * \cos\alpha - W * \cos^2\alpha * \tan\phi - C * b}$$

Determination of the components of equation:

Slide plane angle,	$\alpha =$ 60.1 deg		(Search for Critical Failure Plane)
Location of Tension Crack	a = 0.0 ft		
Length of Wedge,	b = 13.2 ft		
Height of Tension Crack,	$H_c =$ 0.0 ft		
Area of Wedge,	$A = b * (h + H_c) / 2 =$	92.2	ft^2
Weight of Wedge,	$W = A * \gamma_s =$	13547	lbs
Coef. of latteral pressure,	$K_o = 1 - \sin\phi =$	0.41	

$$d = \frac{1/3 * 147 * 0.41 * \tan 36 * (14 * 14 * (0 + 13.2) - 0 * 0 * 0) + 2 * 680 * 92.2}{1.25 * 13547.5 * \sin 60.1 * \cos 60.1 - 13547.5 * \cos 60.1 * \cos 60.1 * \tan 36 - 680 * 13.2}$$

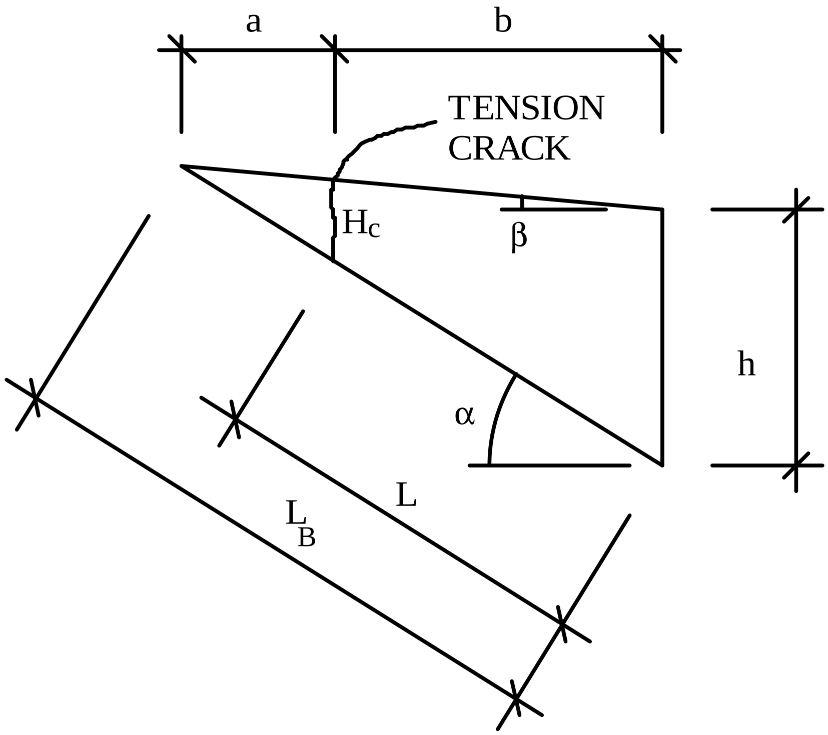
$$d = \frac{163205.6}{-4080.5} = -40.0 \quad \text{ft}$$

The Wedge Is Not Failing

TENSION CRACK LOCATION
(BEDROCK)

DATA:

Soil Density,	γ_s =	147	pcf
Cohesion,	C =	680	psf
Friction Angle,	ϕ =	36	degrees
Surface Angle,	β =	34.0	degrees
Fail. Plane Angle,	α =	60.1	degrees
Height of Cut,	h =	14.0	ft
Factor of Safety,	F.S.=	1.0	



HEIGHT AND LOCATION OF TENSION CRACK:

Total Length of Block,	$L_B = (h * \cos\beta) / (\sin(\alpha - \beta))$ =	26.4	ft
Height of Crack,	$H_c = C / (\gamma_s * \cos\alpha * (\sin\alpha * F.S. - \cos\alpha * \tan\phi))$ =	18.4	ft
Location of Crack,	$a = H_c / (\tan\alpha - \tan\beta)$ =	17.3	ft
Location of Crack,	$b = L_B * \cos\alpha - H_c / (\tan\alpha - \tan\beta)$ =	-4.1	ft
Length of Failure Plane,	$L = b / \cos\alpha$ =	-8.3	ft

No Tension Crack; Block is Not Failing

APPENDIX F

STATIC EQUIVALENT FLUID PRESSURE
BASEMENT RETAINING WALLS
FOR 10 FEET HIGH RETAINING WALL

Wedge No.	Lateral Load from Active Pressure (Single Wedge) (lbs/lf)	Lateral Load from Active Pressure (Accumulated) (lbs/lf)	Equivalent Fluid Pressure psf/ft or pcf
1	1	1	0.01

EFP calculated for H=	10	ft

Total Density, γ_t =	147	pcf	
Saturated Density, γ_s =	147	pcf	
Water Density, γ_w =	62.4	pcf	
Friction Angle, ϕ =	36	degrees	
Cohesion, C =	680	psf	
Surface Angle, β =	0	degrees	
Fail. Plane Angle, α =	57.9	degrees	(Search for Critical Failure Plane)

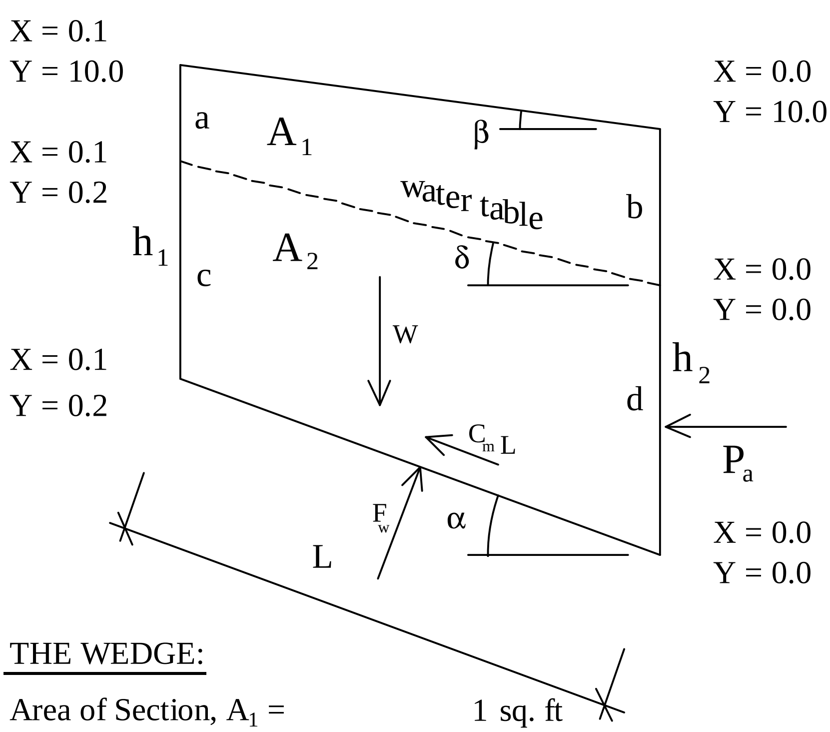
LATERAL LOAD APPLIED ON BLOCK 1

BASEMENT RETAINING WALLS

FOR 10 FEET HIGH RETAINING WALL

DATA:

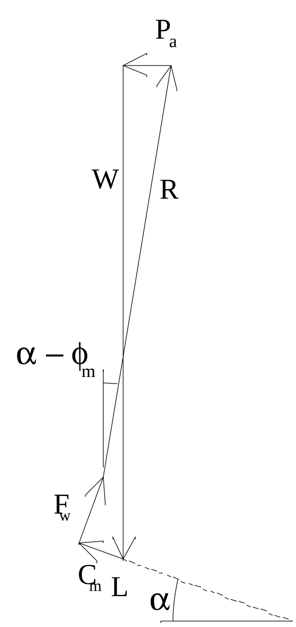
Total Density, γ_t =	147	pcf			
Saturated Density, γ_s =	147	pcf			
Water Density, γ_w =	62.4	pcf			
Friction Angle, ϕ =	36.0	degrees	Mobilized, ϕ_m =	25.8	degrees
Cohesion, C =	680	psf	Mobilized, C_m =	453	psf
Fail. Plane Angle, α =	57.9	degrees			
Surface Angle, β =	0.0	degrees			
Water Table Angle, δ =	57.9	degrees			
Wedge Length, L =	0.2	ft			
Factor of Safety, FS =	1.5				



a =	9.8	ft
b =	10.0	ft
c =	0.0	ft
d =	0.0	ft
h_1=	9.8	ft
h_2=	10.0	ft

THE WEDGE:

Area of Section, A_1 =	1	sq. ft
Area of Section, A_2 =	0	sq. ft
Total Area, A =	1	sq. ft
Weight of Soil, W =	146	lbs/lf
Cohesion, C_mL =	85	lbs/lf
Uplift Force, F_w =	0	lbs/lf



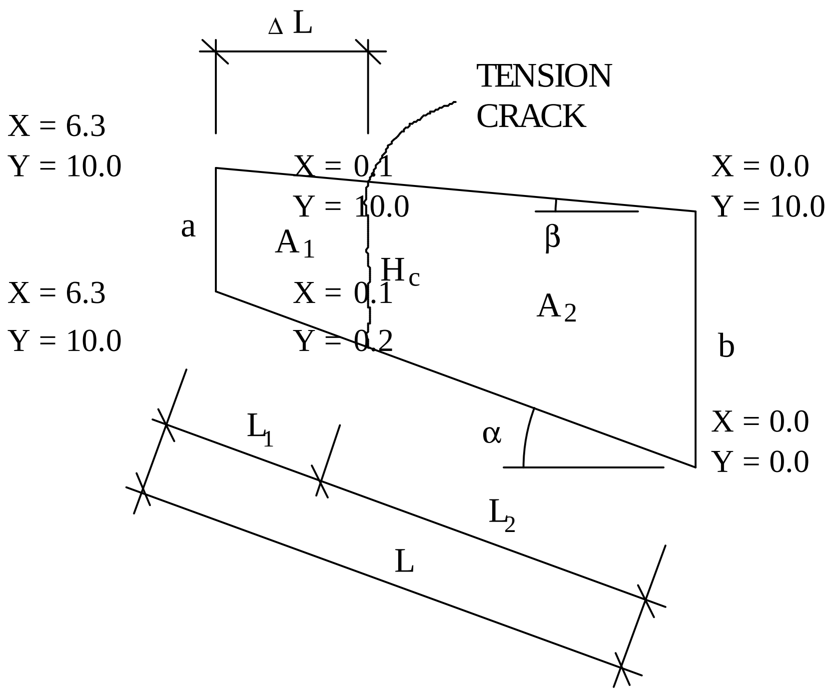
Lateral Load, P_a = 1 lbs/lf

TENSION CRACK LOCATION

BASEMENT RETAINING WALLS

DATA:

Soil Density, γ_t =	147	pcf
Friction Angle, ϕ =	36	degrees
Cohesion, C =	680	psf
Surface Angle, β =	0.0	degrees
Fail. Plane Angle, α =	57.9	degrees
Wedge Length, L =	12	ft
Factor of Safety, F.S.=	1.5	



a =	0.0	ft
b =	10.0	ft
l=	6.3	

HEIGHT AND LOCATION OF TENSION CRACK:

Height of Crack, H_c =	9.8	ft
Location of Crack, ΔL =	6.2	ft

SECTION OF WEDGE ABOVE THE CRACK:

Length of Section, L_1 =	12	ft	Driving Force, W_{D1} =	3,780	lbs
Area of Section, A_1 =	30	sq. ft	Friction, F_{fr1} =	1,721	lbs
Weight of Section, W_1 =	4,461	lbs	Cohesion, CL_1 =	7,897	lbs

Horizontal Projection of Resulting Force, P_1 = -3,101 lbs

SECTION OF WEDGE BELOW THE CRACK:

Length of Section, L_2 =	0	ft	Driving Force, W_{D2} =	123	lbs
Area of Section, A_2 =	1	sq. ft	Friction, F_{fr2} =	56	lbs
Weight of Section, W_2 =	146	lbs	Cohesion, CL_2 =	128	lbs

Horizontal Projection of Resulting Force, P_2 = -32 lbs

PSEUDO-STATIC EQUIVALENT FLUID PRESSURE
BASEMENT RETAINING WALLS
FOR 10 FEET HIGH RETAINING WALL

Wedge No.	Lateral Load from Active Pressure (Single Wedge) (lbs/lf)	Lateral Load from Active Pressure (Accumulated) (lbs/lf)	Equivalent Fluid Pressure psf/ft or pcf
1	-3,394	-3,394	-

EFP calculated for H= 10 ft

Total Density, γ_t =	147	pcf	
Saturated Density, γ_s =	147	pcf	
Water Density, γ_w =	62.4	pcf	
Friction Angle, ϕ =	36	degrees	
Cohesion, C =	680	psf	
Surface Angle, β =	0	degrees	
Fail. Plane Angle, α =	57.3	degrees	(Search for Critical Failure Plane)
Required F.S. =	1		
Seismic Forces	Yes		
Coef. of Horiz. Accel. =	0.345		(PGA_M = 1.035)
Coef. of Vert. Accel. =	0		

NOTE: - The Pseudo-Static Analysis Combines The Earth Pressures
From Static And Seismic Forces

LATERAL LOAD APPLIED ON BLOCK 1

BASEMENT RETAINING WALLS
FOR 10 FEET HIGH RETAINING WALL

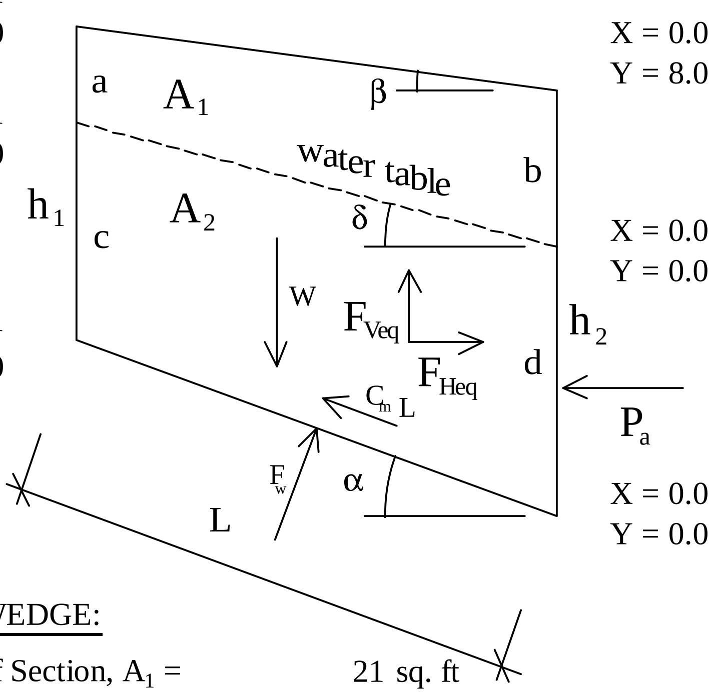
DATA:

Total Density, γ_t =	147	pcf	Coef. of Horiz. Accel. =	0.345	
Saturated Density, γ_s =	147	pcf	Coef. of Vert. Accel. =	0	
Water Density, γ_w =	62.4	pcf			
Friction Angle, ϕ =	36.0	degrees	Mobilized, ϕ_m =	36.0	degrees
Cohesion, C =	680	psf	Mobilized, C_m =	680	psf
Fail. Plane Angle, α =	57.3	degrees			
Surface Angle, β =	0.0	degrees			
Water Table Angle, δ =	57.3	degrees			
Wedge Length, L =	9.5	ft			
Factor of Safety, FS =	1.0				

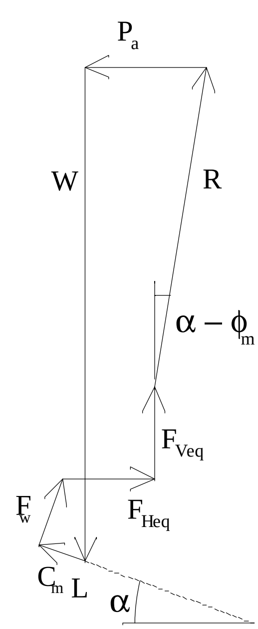
X = 5.1
Y = 8.0

X = 5.1
Y = 8.0

X = 5.1
Y = 8.0



a =	0.0	ft
b =	8.0	ft
c =	0.0	ft
d =	0.0	ft
h_1=	0.0	ft
h_2=	8.0	ft



THE WEDGE:

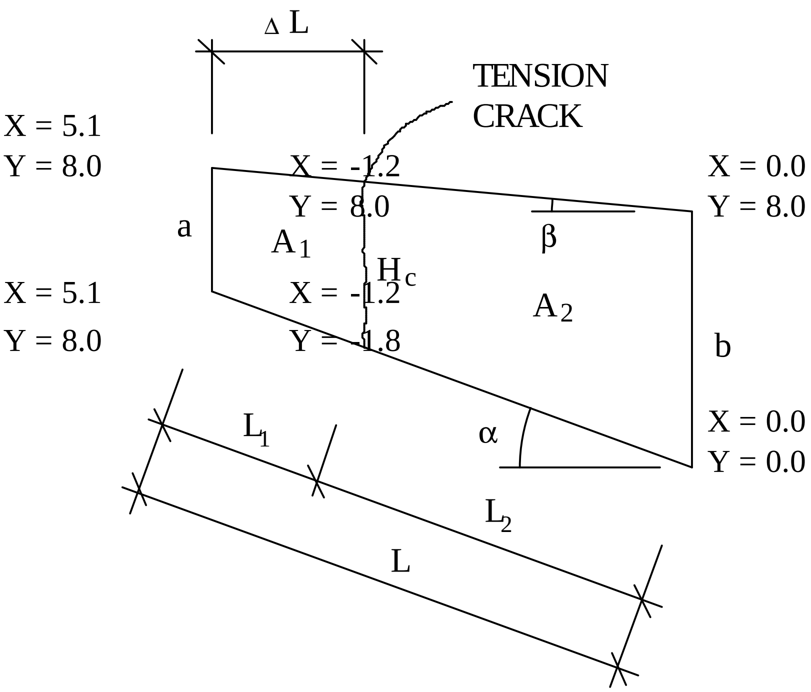
Area of Section, A_1 =	21	sq. ft
Area of Section, A_2 =	0	sq. ft
Total Area, A =	21	sq. ft
Weight of Soil, W =	3,016	lbs/lf
Cohesion, C_mL =	6,462	lbs/lf
Uplift Force, F_w =	0	lbs/lf
Horiz. Seism. Force, F_{Heq}=	1,041	lbs/lf
Vert. Seism. Force, F_{Veq}=	0	lbs/lf

Lateral Load, P_a = -3,394 lbs/lf

TENSION CRACK LOCATION
BASEMENT RETAINING WALLS

DATA:

Soil Density, γ_t =	147	pcf
Friction Angle, ϕ =	36	degrees
Cohesion, C =	680	psf
Surface Angle, β =	0.0	degrees
Fail. Plane Angle, α =	57.3	degrees
Wedge Length, L =	10	ft
Factor of Safety, F.S.=	1.5	



a =	0.0	ft
b =	8.0	ft
l=	5.1	

HEIGHT AND LOCATION OF TENSION CRACK:

Height of Crack, H_c = 9.8 ft — NoTension Crack, Entire Wedge is Stable

Location of Crack, ΔL = 6.3 ft

SECTION OF WEDGE ABOVE THE CRACK:

Length of Section, L_1 =	12	ft	Driving Force, W_{D1} =	3,845	lbs
Area of Section, A_1 =	31	sq. ft	Friction, F_{fr1} =	1,791	lbs
Weight of Section, W_1 =	4,567	lbs	Cohesion, CL_1 =	7,952	lbs

Horizontal Projection of Resulting Force, P_1 = -3,184 lbs

SECTION OF WEDGE BELOW THE CRACK:

Length of Section, L_2 =	-2	ft	Driving Force, W_{D2} =	-1,306	lbs
Area of Section, A_2 =	-11	sq. ft	Friction, F_{fr2} =	-608	lbs
Weight of Section, W_2 =	-1,551	lbs	Cohesion, CL_2 =	-1,490	lbs

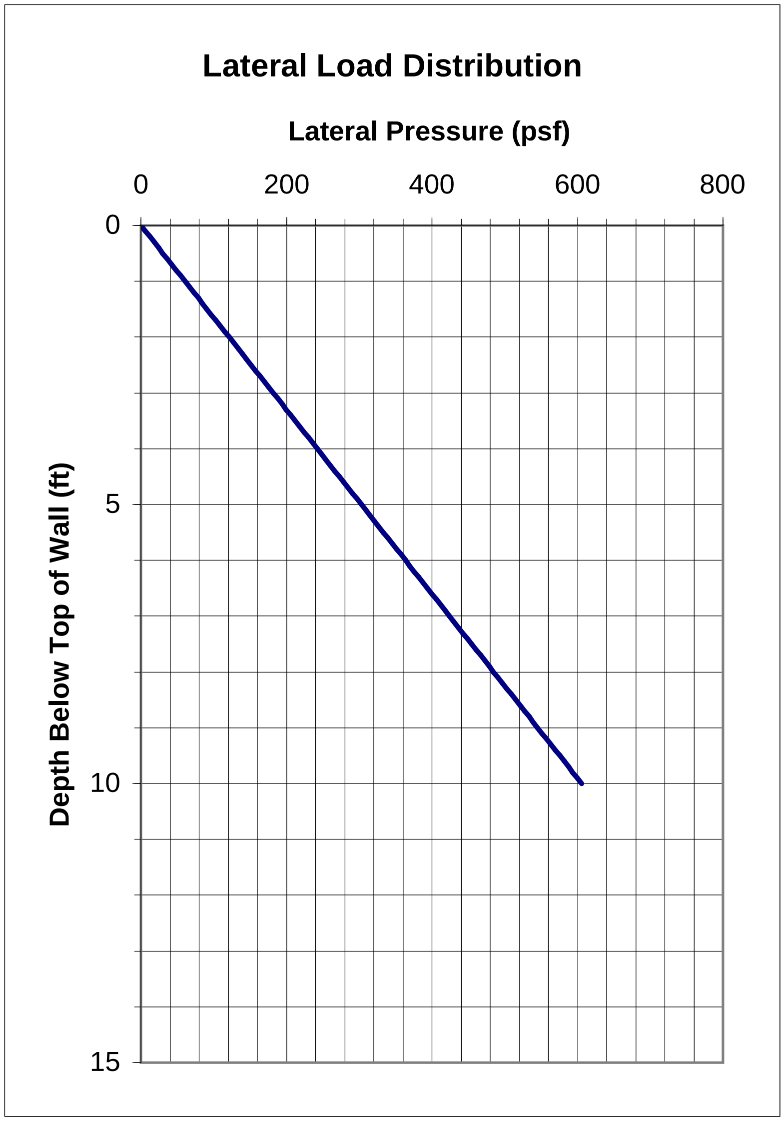
Horizontal Projection of Resulting Force, P_2 = 428 lbs

LATERAL LOAD DISTRIBUTION FOR RETAINING WALL
RESTRAINED 10 FOOT HIGH BASEMENT WALLS

Initial Input:

Soil and Retaining Wall Data:

Wall Type - Braced		B	
Height of Wall	H =	10	ft
Total Density	γ_t =	147	pcf
Saturated Density	γ_s =	147	pcf
Cohesion	C =	680	psf
Friction Angle	ϕ =	36	deg
Depth of Water Table	d_w =	30	ft
Poisson Ratio	μ =	0.292	



Distributed Surcharge Data:

Distributed Load	q =	0	psf
Distance from Wall	L_1 =	0.0	ft
Width of Load	B_1 =	0.0	ft

Concentrated Load Data:

Concentr. Load 1	P_1 =	0	lb
Concentr. Load 2	P_2 =	0	lb
Distance from Wall	L_2 =	0.0	ft
Dist. between Loads	B_2 =	0.0	ft

Adjacent Footing Data:

Linear Load on Footing	Q =	0	plf
Distance from Wall	L_3 =	0	ft
Width of Footing	B_3 =	0.0	ft
Depth of Footing	D =	0.0	ft

Eqiuvalent Fluid Pressure: 61.2 pcf
Total Force Acting on Wall: 3060 lb/ft
Point of Application: 6.7 ft Below Top of Wall
Pressure at Poin 'A': 103.01 psf @ 1.7 ft Below Top of Wall
Pressure at Poin 'B': 302.98 psf @ 5 ft Below Top of Wall
Relative Maximum: 605.96 psf @ 10 ft Below Top of Wall

STATIC EQUIVALENT FLUID PRESSURE
YARD RETAINING WALLS WITH BACKFILL
FOR 8 FEET HIGH RETAINING WALL

Wedge No.	Lateral Load from Active Pressure (Single Wedge) (lbs/lf)	Lateral Load from Active Pressure (Accumulated) (lbs/lf)	Equivalent Fluid Pressure psf/ft or pcf
1	1,056	1,056	33.0

EFP calculated for H=	8	ft

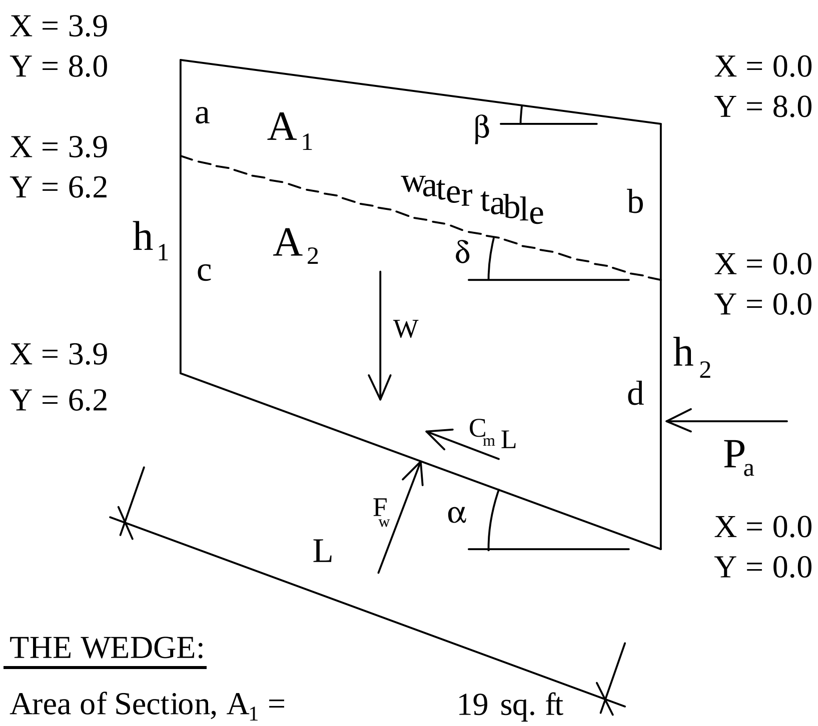
Total Density, γ_t =	137	pcf	
Saturated Density, γ_s =	137	pcf	
Water Density, γ_w =	62.4	pcf	
Friction Angle, ϕ =	35	degrees	
Cohesion, C =	120	psf	
Surface Angle, β =	0	degrees	
Fail. Plane Angle, α =	57.5	degrees	(Search for Critical Failure Plane)

LATERAL LOAD APPLIED ON BLOCK 1

YARD RETAINING WALLS WITH BACKFILL
FOR 8 FEET HIGH RETAINING WALL

DATA:

Total Density, γ_t =	137	pcf			
Saturated Density, γ_s =	137	pcf			
Water Density, γ_w =	62.4	pcf			
Friction Angle, ϕ =	35.0	degrees	Mobilized, ϕ_m =	25.0	degrees
Cohesion, C =	120	psf	Mobilized, C_m =	80	psf
Fail. Plane Angle, α =	57.5	degrees			
Surface Angle, β =	0.0	degrees			
Water Table Angle, δ =	57.5	degrees			
Wedge Length, L =	7.3	ft			
Factor of Safety, FS =	1.5				

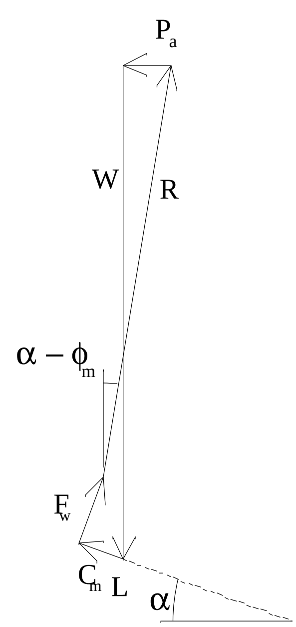


a =	1.8	ft
b =	8.0	ft
c =	0.0	ft
d =	0.0	ft
h_1=	1.8	ft
h_2=	8.0	ft

THE WEDGE:

Area of Section, A_1 =	19	sq. ft
Area of Section, A_2 =	0	sq. ft
Total Area, A =	19	sq. ft
Weight of Soil, W =	2,645	lbs/lf
Cohesion, $C_m L$ =	585	lbs/lf
Uplift Force, F_w =	0	lbs/lf

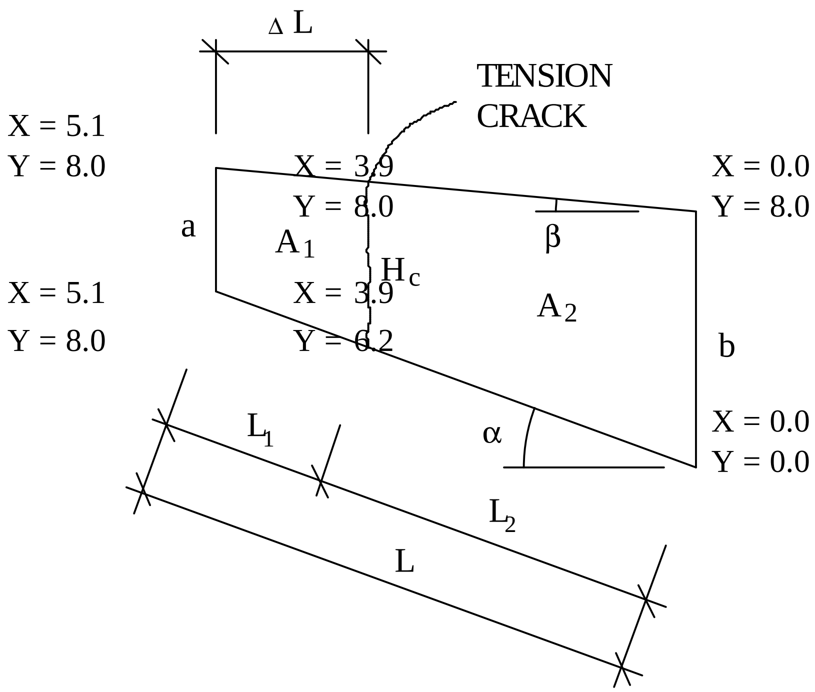
Lateral Load, P_a = 1,056 lbs/lf



TENSION CRACK LOCATION
YARD RETAINING WALLS WITH BACKFILL

DATA:

Soil Density, γ_t =	137	pcf
Friction Angle, ϕ =	35	degrees
Cohesion, C =	120	psf
Surface Angle, β =	0.0	degrees
Fail. Plane Angle, α =	57.5	degrees
Wedge Length, L =	9	ft
Factor of Safety, F.S.=	1.5	



a =	0.0	ft
b =	8.0	ft
l=	5.1	

HEIGHT AND LOCATION OF TENSION CRACK:

Height of Crack, H_c =	1.8	ft
Location of Crack, ΔL =	1.2	ft

SECTION OF WEDGE ABOVE THE CRACK:

Length of Section, L_1 =	2	ft	Driving Force, W_{D1} =	124	lbs
Area of Section, A_1 =	1	sq. ft	Friction, F_{fr1} =	55	lbs
Weight of Section, W_1 =	147	lbs	Cohesion, CL_1 =	261	lbs

Horizontal Projection of Resulting Force, P_1 = -103 lbs

SECTION OF WEDGE BELOW THE CRACK:

Length of Section, L_2 =	7	ft	Driving Force, W_{D2} =	2,231	lbs
Area of Section, A_2 =	19	sq. ft	Friction, F_{fr2} =	995	lbs
Weight of Section, W_2 =	2,645	lbs	Cohesion, CL_2 =	877	lbs

Horizontal Projection of Resulting Force, P_2 = 193 lbs

PSEUDO-STATIC EQUIVALENT FLUID PRESSURE
YARD RETAINING WALLS WITH BACKFILL
FOR 8 FEET HIGH RETAINING WALL

Wedge No.	Lateral Load from Active Pressure (Single Wedge) (lbs/lf)	Lateral Load from Active Pressure (Accumulated) (lbs/lf)	Equivalent Fluid Pressure psf/ft or pcf
1	1,318	1,318	41.2

EFP calculated for H= 8 ft

Total Density, γ_t =	137	pcf	
Saturated Density, γ_s =	137	pcf	
Water Density, γ_w =	62.4	pcf	
Friction Angle, ϕ =	35	degrees	
Cohesion, C =	120	psf	
Surface Angle, β =	0	degrees	
Fail. Plane Angle, α =	49.1	degrees	(Search for Critical Failure Plane)
Required F.S. =	1		
Seismic Forces	Yes		
Coef. of Horiz. Accel. =	0.345		(PGA_M = 1.035)
Coef. of Vert. Accel. =	0		

NOTE: - The Pseudo-Static Analysis Combines The Earth Pressures
From Static And Seismic Forces

LATERAL LOAD APPLIED ON BLOCK 1

YARD RETAINING WALLS WITH BACKFILL
FOR 8 FEET HIGH RETAINING WALL

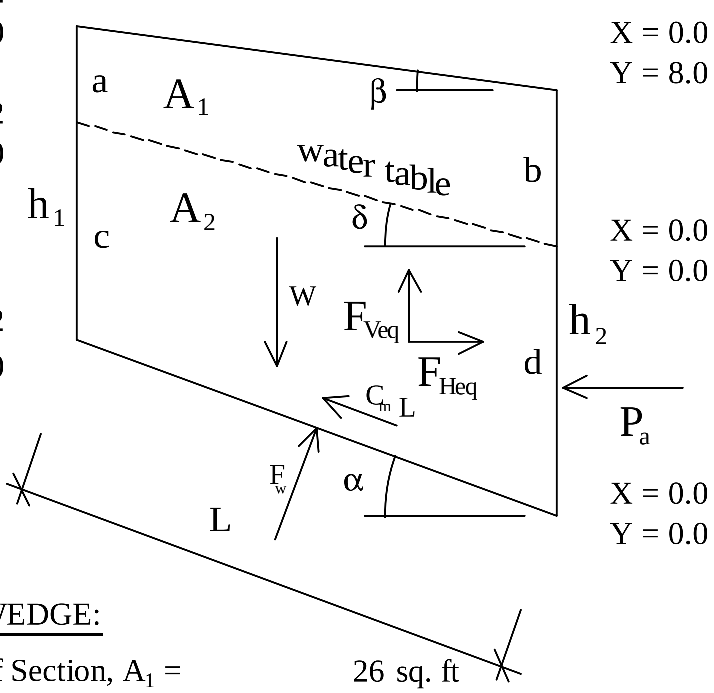
DATA:

Total Density, γ_t =	137	pcf	Coef. of Horiz. Accel. =	0.345	
Saturated Density, γ_s =	137	pcf	Coef. of Vert. Accel. =	0	
Water Density, γ_w =	62.4	pcf			
Friction Angle, ϕ =	35.0	degrees	Mobilized, ϕ_m =	35.0	degrees
Cohesion, C =	120	psf	Mobilized, C_m =	120	psf
Fail. Plane Angle, α =	49.1	degrees			
Surface Angle, β =	0.0	degrees			
Water Table Angle, δ =	49.1	degrees			
Wedge Length, L =	8.0	ft			
Factor of Safety, FS =	1.0				

X = 5.2
Y = 8.0

X = 5.2
Y = 6.0

X = 5.2
Y = 6.0

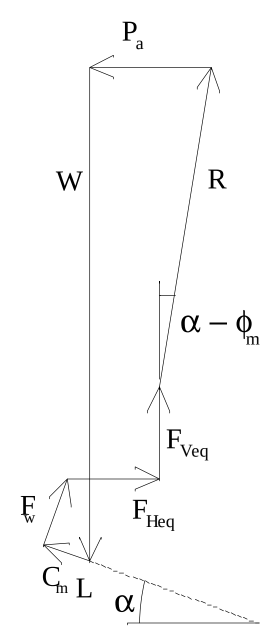


X = 0.0
Y = 8.0

X = 0.0
Y = 0.0

X = 0.0
Y = 0.0

a =	2.0	ft
b =	8.0	ft
c =	0.0	ft
d =	0.0	ft
h_1=	2.0	ft
h_2=	8.0	ft



THE WEDGE:

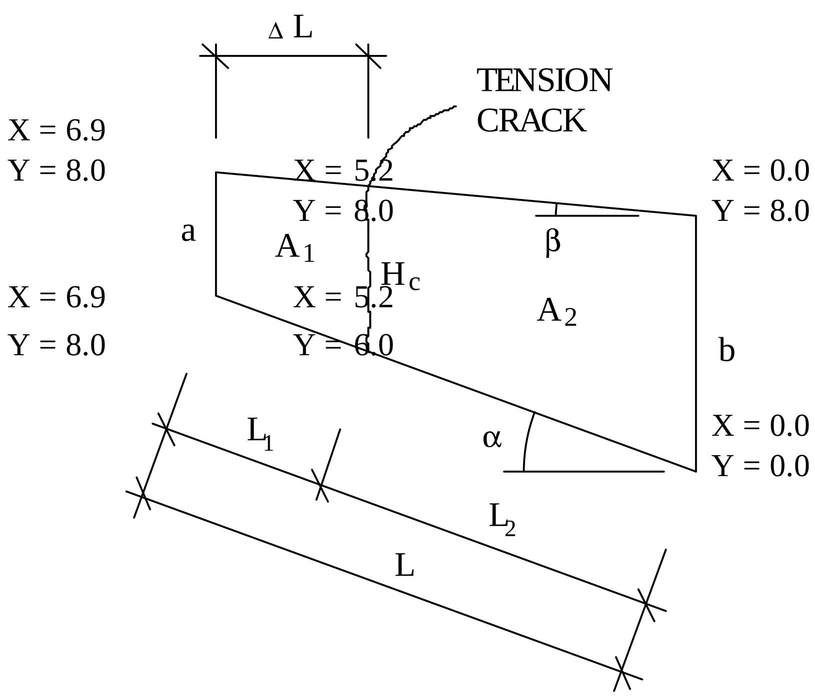
Area of Section, A_1 =	26	sq. ft
Area of Section, A_2 =	0	sq. ft
Total Area, A =	26	sq. ft
Weight of Soil, W =	3,561	lbs/lf
Cohesion, C_mL =	955	lbs/lf
Uplift Force, F_w =	0	lbs/lf
Horiz. Seism. Force, F_{Heq}=	1,229	lbs/lf
Vert. Seism. Force, F_{Veq}=	0	lbs/lf
Lateral Load, P_a =	**1,318**	**lbs/lf**

TENSION CRACK LOCATION

YARD RETAINING WALLS WITH BACKFILL

DATA:

Soil Density, γ_t =	137	pcf
Friction Angle, ϕ =	35	degrees
Cohesion, C =	120	psf
Surface Angle, β =	0.0	degrees
Fail. Plane Angle, α =	49.1	degrees
Wedge Length, L =	11	ft
Factor of Safety, F.S.=	1.5	



a =	0.0	ft
b =	8.0	ft
l=	6.9	

HEIGHT AND LOCATION OF TENSION CRACK:

Height of Crack, H_c =	2.0	ft
Location of Crack, ΔL =	1.7	ft

SECTION OF WEDGE ABOVE THE CRACK:

Length of Section, L_1 =	3	ft	Driving Force, W_{D1} =	176	lbs
Area of Section, A_1 =	2	sq. ft	Friction, F_{fr1} =	106	lbs
Weight of Section, W_1 =	232	lbs	Cohesion, CL_1 =	314	lbs

Horizontal Projection of Resulting Force, P_1 = -160 lbs

SECTION OF WEDGE BELOW THE CRACK:

Length of Section, L_2 =	8	ft	Driving Force, W_{D2} =	2,693	lbs
Area of Section, A_2 =	26	sq. ft	Friction, F_{fr2} =	1,632	lbs
Weight of Section, W_2 =	3,561	lbs	Cohesion, CL_2 =	955	lbs

Horizontal Projection of Resulting Force, P_2 = 69 lbs

STATIC EQUIVALENT FLUID PRESSURE
RETAINING WALL TO THE WEST OF RESIDENCE
FOR 14 FEET HIGH RETAINING WALL

Wedge No.	Lateral Load from Active Pressure (Single Wedge) (lbs/lf)	Lateral Load from Active Pressure (Accumulated) (lbs/lf)	Equivalent Fluid Pressure psf/ft or pcf
1	900	900	9.2

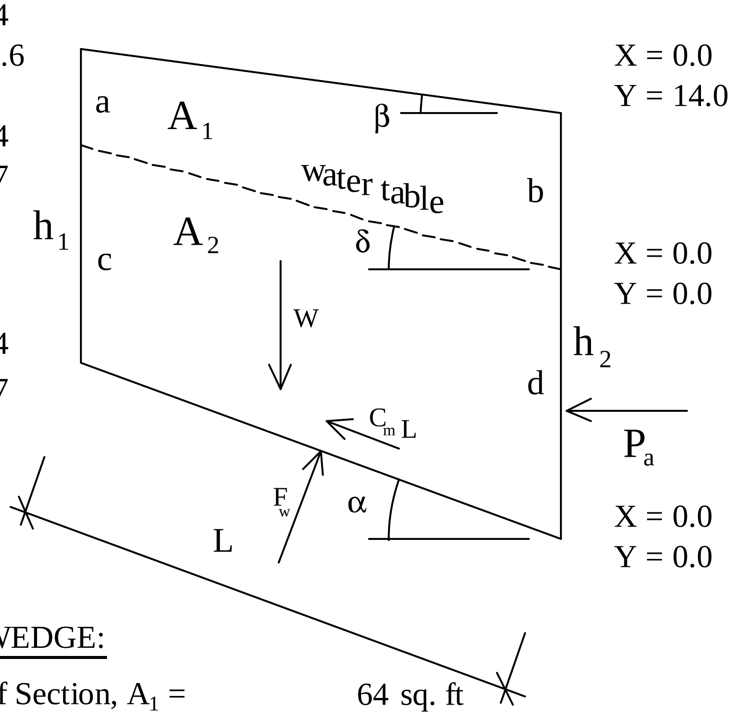
EFP calculated for H= 14 ft

Total Density, γ_t =	147	pcf	
Saturated Density, γ_s =	147	pcf	
Water Density, γ_w =	62.4	pcf	
Friction Angle, ϕ =	36	degrees	
Cohesion, C =	680	psf	
Surface Angle, β =	34	degrees	
Fail. Plane Angle, α =	55.2	degrees	(Search for Critical Failure Plane)

LATERAL LOAD APPLIED ON BLOCK 1
RETAINING WALL TO THE WEST OF RESIDENCE
FOR 14 FEET HIGH RETAINING WALL

DATA:

Total Density, γ_t =	147	pcf			
Saturated Density, γ_s =	147	pcf			
Water Density, γ_w =	62.4	pcf			
Friction Angle, ϕ =	36.0	degrees	Mobilized, ϕ_m =	25.8	degrees
Cohesion, C =	680	psf	Mobilized, C_m =	453	psf
Fail. Plane Angle, α =	55.2	degrees			
Surface Angle, β =	34.0	degrees			
Water Table Angle, δ =	55.2	degrees			
Wedge Length, L =	9.4	ft			
Factor of Safety, FS =	1.5				

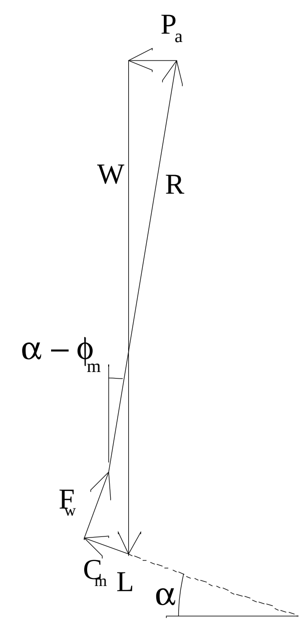


a =	9.9	ft
b =	14.0	ft
c =	0.0	ft
d =	0.0	ft
h_1=	9.9	ft
h_2=	14.0	ft

THE WEDGE:

Area of Section, A_1 =	64	sq. ft
Area of Section, A_2 =	0	sq. ft
Total Area, A =	64	sq. ft
Weight of Soil, W =	9,410	lbs/lf
Cohesion, C_mL =	4,246	lbs/lf
Uplift Force, F_w =	0	lbs/lf

Lateral Load, P_a = 900 lbs/lf

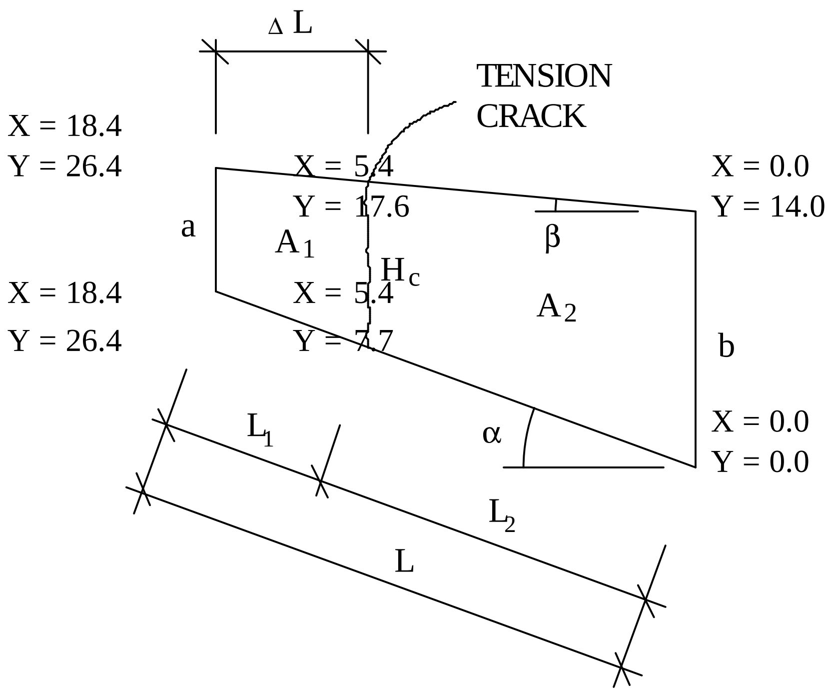


TENSION CRACK LOCATION

RETAINING WALL TO THE WEST OF RESIDENCE

DATA:

Soil Density, γ_t =	147	pcf
Friction Angle, ϕ =	36	degrees
Cohesion, C =	680	psf
Surface Angle, β =	34.0	degrees
Fail. Plane Angle, α =	55.2	degrees
Wedge Length, L =	32	ft
Factor of Safety, F.S.=	1.5	



a =	0.0	ft
b =	14.0	ft
l=	18.4	

HEIGHT AND LOCATION OF TENSION CRACK:

Height of Crack, H_c =	9.9	ft
Location of Crack, ΔL =	13.0	ft

SECTION OF WEDGE ABOVE THE CRACK:

Length of Section, L_1 =	23	ft	Driving Force, W_{D1} =	7,795	lbs
Area of Section, A_1 =	65	sq. ft	Friction, F_{fr1} =	3,943	lbs
Weight of Section, W_1 =	9,498	lbs	Cohesion, CL_1 =	15,500	lbs

Horizontal Projection of Resulting Force, P_1 = -6,655 lbs

SECTION OF WEDGE BELOW THE CRACK:

Length of Section, L_2 =	9	ft	Driving Force, W_{D2} =	7,723	lbs
Area of Section, A_2 =	64	sq. ft	Friction, F_{fr2} =	3,906	lbs
Weight of Section, W_2 =	9,410	lbs	Cohesion, CL_2 =	6,369	lbs

Horizontal Projection of Resulting Force, P_2 = -1,459 lbs

PSEUDO-STATIC EQUIVALENT FLUID PRESSURE
RETAINING WALL TO THE WEST OF RESIDENCE
FOR 10 FEET HIGH RETAINING WALL

Wedge No.	Lateral Load from Active Pressure (Single Wedge) (lbs/lf)	Lateral Load from Active Pressure (Accumulated) (lbs/lf)	Equivalent Fluid Pressure psf/ft or pcf
1	1,276	1,276	25.5

EFP calculated for H= 10 ft

Total Density, γ_t =	147	pcf	
Saturated Density, γ_s =	147	pcf	
Water Density, γ_w =	62.4	pcf	
Friction Angle, ϕ =	36	degrees	
Cohesion, C =	680	psf	
Surface Angle, β =	34	degrees	
Fail. Plane Angle, α =	49.1	degrees	(Search for Critical Failure Plane)
Required F.S. =	1		
Seismic Forces	Yes		
Coef. of Horiz. Accel. =	0.345		(PGA_M = 1.035)
Coef. of Vert. Accel. =	0		

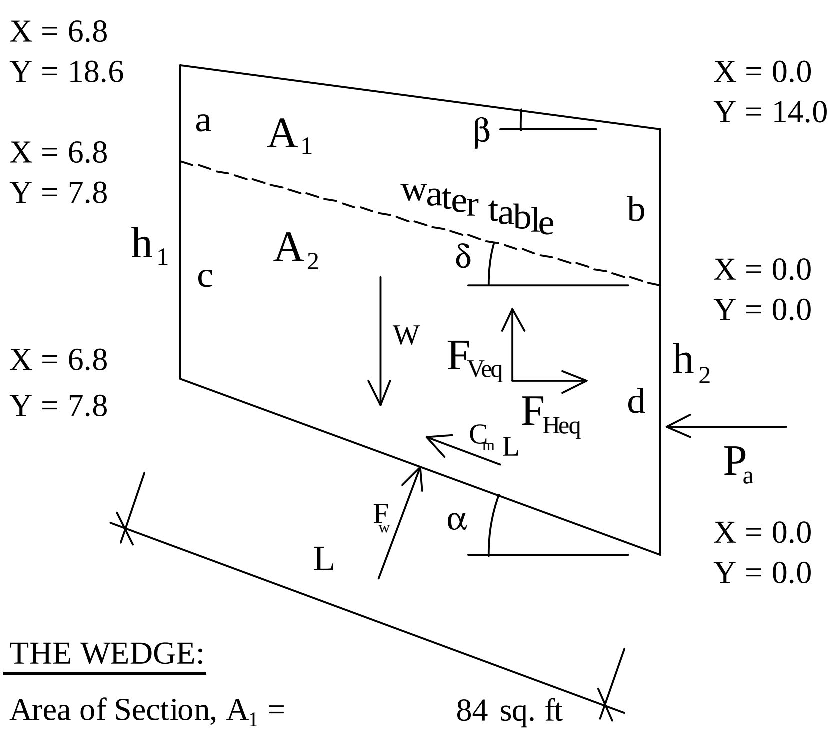
NOTE: - The Pseudo-Static Analysis Combines The Earth Pressures From Static And Seismic Forces

LATERAL LOAD APPLIED ON BLOCK 1

RETAINING WALL TO THE WEST OF RESIDENCE
FOR 10 FEET HIGH RETAINING WALL

DATA:

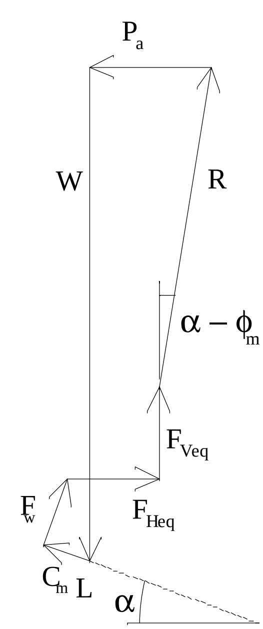
Total Density, γ_t =	147	pcf	Coef. of Horiz. Accel. =	0.345	
Saturated Density, γ_s =	147	pcf	Coef. of Vert. Accel. =	0	
Water Density, γ_w =	62.4	pcf			
Friction Angle, ϕ =	36.0	degrees	Mobilized, ϕ_m =	36.0	degrees
Cohesion, C =	680	psf	Mobilized, C_m =	680	psf
Fail. Plane Angle, α =	49.1	degrees			
Surface Angle, β =	34.0	degrees			
Water Table Angle, δ =	49.1	degrees			
Wedge Length, L =	10.4	ft			
Factor of Safety, FS =	1.0				



a =	10.7	ft
b =	14.0	ft
c =	0.0	ft
d =	0.0	ft
h_1=	10.7	ft
h_2=	14.0	ft

THE WEDGE:

Area of Section, A_1 =	84	sq. ft
Area of Section, A_2 =	0	sq. ft
Total Area, A =	84	sq. ft
Weight of Soil, W =	12,372	lbs/lf
Cohesion, C_mL =	7,063	lbs/lf
Uplift Force, F_w =	0	lbs/lf
Horiz. Seism. Force, F_{Heq}=	4,268	lbs/lf
Vert. Seism. Force, F_{Veq}=	0	lbs/lf
Lateral Load, P_a =	**1,276**	**lbs/lf**

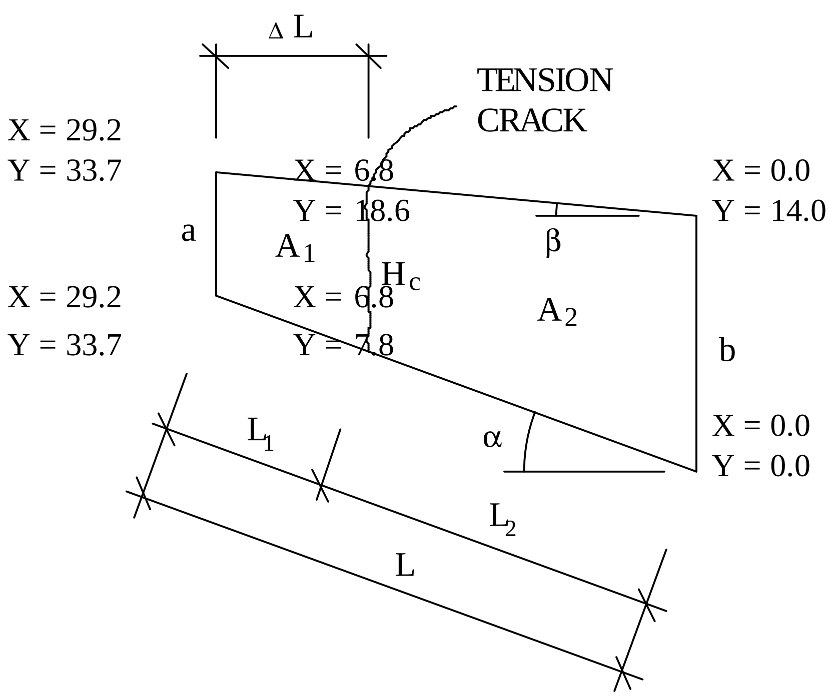


TENSION CRACK LOCATION

RETAINING WALL TO THE WEST OF RESIDENCE

DATA:

Soil Density, γ_t =	147	pcf
Friction Angle, ϕ =	36	degrees
Cohesion, C =	680	psf
Surface Angle, β =	34.0	degrees
Fail. Plane Angle, α =	49.1	degrees
Wedge Length, L =	45	ft
Factor of Safety, F.S.=	1.5	



a =	0.0	ft
b =	14.0	ft
l=	29.2	

HEIGHT AND LOCATION OF TENSION CRACK:

Height of Crack, H_c =	10.7	ft
Location of Crack, ΔL =	22.4	ft

SECTION OF WEDGE ABOVE THE CRACK:

Length of Section, L_1 =	34	ft	Driving Force, W_{D1} =	13,375	lbs
Area of Section, A_1 =	120	sq. ft	Friction, F_{fr1} =	8,424	lbs
Weight of Section, W_1 =	17,701	lbs	Cohesion, CL_1 =	23,276	lbs

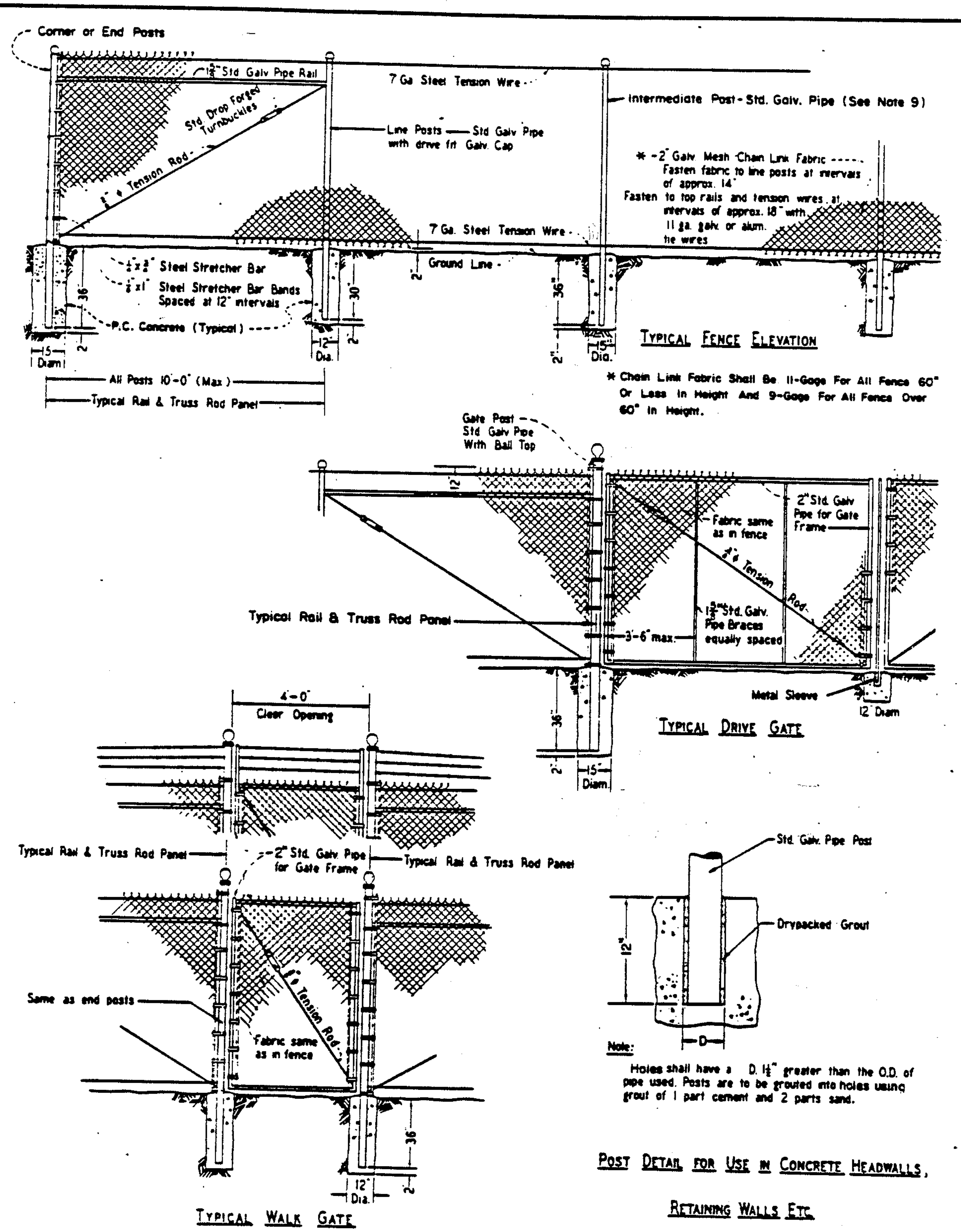
Horizontal Projection of Resulting Force, P_1 = -12,004 lbs

SECTION OF WEDGE BELOW THE CRACK:

Length of Section, L_2 =	10	ft	Driving Force, W_{D2} =	9,349	lbs
Area of Section, A_2 =	84	sq. ft	Friction, F_{fr2} =	5,888	lbs
Weight of Section, W_2 =	12,372	lbs	Cohesion, CL_2 =	7,063	lbs

Horizontal Projection of Resulting Force, P_2 = -2,360 lbs

APPENDIX G



REARRANGED BY R. SADLEIR 2-63
CHECKED BY J. MONTAPERTO 2-63

LOS ANGELES COUNTY ROAD DEPARTMENT

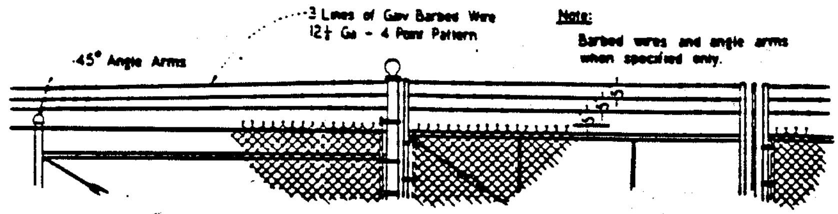
CHAIN LINK FENCE

STANDARD PLAN

88-01

APPROVED I. L. Mosher 1 Dec 67

ROAD COMMISSIONER DATE



TYPICAL BARBED WIRE FENCE & GATES

GENERAL NOTES

1. All posts shall be Standard Galvanized Pipe and shall conform to the following requirements:

FENCES 72" and OVER	PIPE Size - inches	PIPE lbs./lin. ft.
End, Corner, and Intermediate Posts	2-1/2	5.79
Line Posts	2	3.65
Drive Gate Posts — Single Opening — up to 6'	2-1/2	5.79
" " " " " 6' to 13'	3-1/2	9.11
Drive Gate Posts — Double Opening — up to 12'	2-1/2	5.79
" " " " " 12' to 26'	3-1/2	9.11
FENCES LESS THAN 72"	**PIPE Size - inches**	**PIPE lbs./lin. ft.**
End, Corner, and Intermediate Posts	2	3.65
Line Posts	1-1/2	2.72
Drive Gate Posts — Single Opening — up to 6'	2	3.65
" " " " " 6' to 13'	3-1/2	9.11
" " " " " 13' to 18'	6	18.97
" " " " " over 18'	8	24.70
Drive Gate Posts — Double Opening — up to 12'	2	3.65
" " " " " 12' to 26'	3-1/2	9.11
" " " " " 26' to 36'	6	18.97
" " " " " over 36'	8	24.70
All Slide Gates With Leaves Longer Than 6'	3-1/2	9.11

2. All posts shall be fitted with galvanized cast iron caps.

3. All walk and drive gates shall be hung by two malleable iron hinges. The bottom hinge to be ball and socket type.

4. Chain link fence fabric shall have twisted and barbed finish on both edges for widths of 60" and over.

5. Fabric less than 60" wide shall have knuckled finish on the top edge and twisted and barbed finish on the bottom edge.

6. Chain link fence fabric shall be fastened on the side of the posts designated by the engineer.

7. Gates shall have a full height locking bar, with not fewer than two sets of locking prongs, and a pair of perforated lugs, or equivalent device for installation of a padlock.

8. Walk gates shall be provided with a combination steel or malleable iron catch and locking attachment approved by the engineer.

9. Intermediate posts are to be braced and trussed to adjacent line posts at 300' intervals along fence line and at alignment deflection angles of 15° or more.

10. All materials and fittings shall be new and all ferrous materials shall be galvanized.

REARRANGED BY: R. SADLEIR, 2-63. CHECKED BY: J. MONTAPERTO, 2-63.
REV. 1-69: CHANGED HEADING FOR SECOND COLUMN IN GENERAL NOTE NO. 1.
REV 12-75: REV GENERAL NOTE NO. 1 - PIPE SIZE.

LOS ANGELES COUNTY ROAD DEPARTMENT

CHAIN LINK FENCE

STANDARD PLAN

88-01

REVISED 12-75