GeoConcepts INC

Geology - Geotechnical Engineering

PRELIMINARY GEOLOGIC AND GEOTECHNICAL ENGINEERING INVESTIGATION

Proposed Fifteen-Lot Subdivision

APN: 2571-001-027

Day Street

Tujunga, California

for

Iris and Crown Investments, LLC

c/o Villa Nova Developing

8209A Foothill Blvd., Suite 700

Sunland, California 91040

Project 2793

November 18, 2014

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INTRODUCTION

This report presents the results of a Preliminary Geologic and Geotechnical Engineering Investigation on a portion of the subject property. The purpose of this investigation has been to ascertain the subsurface conditions pertaining to the proposed project. The work performed for the project included reconnaissance mapping, description of earth materials, determining geologic structure, obtaining representative samples of earth materials, laboratory testing, engineering analyses, and preparation of this report. Results of the project include findings, conclusions, and appropriate recommendations.

SCOPE

The scope of this investigation included the following:

- Review of preliminary plans by Southland Civil Engineering & Survey, LLP.
- Review of ten (10) seismic trench explorations. Explorations were backfilled with the excavated materials.
- Preparation of the enclosed Geologic Map and Cross Sections, (see Appendix I).
- Sampling of representative earth materials, laboratory testing, and engineering analyses (see Appendix II).
- Review of referenced materials, previously prepared reports by this office, and available public reports at the City of Los Angeles (see Appendix V).
- Presentation of findings, conclusions, and recommendations for the proposed project.

Southland Civil Engineering & Survey, LLP prepared the topographic base map and preliminary grading plans utilized in this investigation. It consists of one sheet drawn to a scale of one-inch equals fifty feet and dated October 3, 2012. The original survey was performed by Land Design Consultants and dated February 2, 2004.

The scope of this investigation is limited to the project area explored as depicted on the Geologic Map. This report is not a comprehensive evaluation of the entire property. This report has not been prepared for use by other parties or for purposes other than the proposed project. GeoConcepts, Inc. should be consulted to determine if additional work is required when our work is used by others or if the scope of the project has changed. If the project is delayed for more than one year, this office should be contacted to verify the current site conditions and to prepare an update report.

PROPOSED DEVELOPMENT

It is our understanding that the subject site will be subdivided into (15) lots for residential development. At this time, (13) two-story single family residences are proposed. Grading and retaining walls will be required to create building pads. Additionally, two detention basins and associated grading are proposed within the natural drainage channels that are northeast of the residential development. Anticipated foundations will range from 1 to 2 kips per lineal foot and 20-40 kips for column foundations. The proposed development is depicted on the enclosed Geologic Map and Cross-Sections.

Grading will consist of conventional cut and fill methods for the building pads and the detention basins. Final grading and building plans have not been prepared and await the conclusions and recommendations of this investigation. These plans should be reviewed by GeoConcepts, Inc. to ensure that our recommendations have been followed.

SITE DESCRIPTION

Location and Description

Access to the property is via Day Street from Amanita Avenue (see Location Map). The subject site consists of a roughly triangular shaped parcel trending lengthwise to the northwest along Day Street from Amanita Avenue to just beyond Pali Avenue. The site is vacant and generally unimproved although significant grading has occurred across the southwestern portion of the subject site resulting in the current topography. Adjacent sites at the northwest corner of the property have been developed with two story single family residences. Adjacent properties to the northeast and southeast are vacant and generally unimproved. The subject site is bound along the southwest by Day Street.

The roughly graded areas have a light growth of vegetation consisting of low grasses and shrubs. The northern portions of the project site area display natural topography and the vegetation in this area is moderately dense on the ascending slope consisting of grasses, shrubs, and few trees.

Topography

Topographically, the property is situated across the southern flank of the San Gabriel Mountains. The property essentially consists of a narrow, generally level cut pad area across the southwestern portion of the site at the approximate street grade level. Steep ascending cut slopes bound the northeastern portions of the graded flat pad. The cut slopes ascend directly from the street level to the northeast. Two natural drainage channels bisect the project site area flowing from the northeast toward the southwest and two spur ridges terminate at the northern portion of the project site area. The flat area along the southern portion of the site is the result of grading/cutting at the toe of the slope resulting in an abrupt termination of the drainage channels and ridge lines. Maximum topographic relief on-site is about (90) feet. Ascending natural slopes are located at the rear of the property on the northeastern portion of the site. The cut slopes ascend at a general gradient of about 1:1 to ¾:1 (horizontal to vertical). Details of the topography are depicted on the Location Map and Geologic Map in Appendix I.

Drainage

Surface water at the site consists of direct precipitation onto the property and runoff from surrounding slopes to the northeast. Much of this water drains as sheet flow down descending slopes to low-lying areas, offsite and/or to the street. No area drains and/or subdrain outlet pipes were observed on the property.

Groundwater

No active surface groundwater seeps or springs were observed on the subject site. The subsurface exploration did not encounter groundwater to a depth of (12) feet. The depth to groundwater, when encountered in the explorations, is only valid for the date of exploration. Seasonal fluctuations of groundwater levels may occur by varying amounts of rainfall, irrigation and recharge.

FIELD EXPLORATION

The scope of the field exploration was developed based on the preliminary information of the proposed development available at the time of the exploration, a review of geologic literature, and a preliminary geologic reconnaissance of the subject site. The locations of the explorations are depicted on the Geologic Map. The field exploration was limited to the southern portions of the subject property and was limited by underground utilities along the southwest perimeter of the property.

The original field exploration of the site was conducted on April, 2004 (Trenches 1 through 6). Additional explorations were conducted on November 13 and 14, 2013, September 12, 2014, and October 7, 2014 (Trenches A through D) in order to verify the reported field conditions and to investigate additional areas on the project site. The earliest explorations were mapped by a representative of this office. The latter explorations were mapped by John Helms CEG and by a representative of this office (refer to Exploration Logs). Subsurface exploration was performed by a conventional backhoe excavating trenches into the underlying earth materials. The trenches excavated are oriented roughly north-south and were excavated utilizing a 3-foot wide bucket to depths ranging up to about (12) feet below the ground surface. Prior to the geologic logging, the trench walls were cleaned to obtain a fresh exposure of the geologic units. Horizontal and vertical stationing was established using a line level and tape measure. All explorations were backfilled and tamped upon completion of down-hole observation. However, some settlement within exploration areas should be anticipated.

Detailed descriptions of the geologic materials encountered during the field exploration are provided in the Exploration Logs and Appendix I.

Undisturbed and bulk samples representative of the earth materials were obtained and transported to our laboratory. Undisturbed Modified California (MC) samples were obtained within the explorations through the use of a thin-walled steel sampler pushed using the backhoe bucket. MC samples were retained in brass rings of two and one-half inches (2½") in diameter and one inch (1") in height. SPT samples were retained in brass tubes of one and one-half inches (1½") in diameter and six inches (6") in height. The samples were transported in moisture tight containers. The results of the laboratory testing and a summary of the test procedures are included within Appendix II.

SUMMARY OF FINDINGS

Previous Work

The subject property is vacant but appears to have been roughly graded. No geology and/or geotechnical reports were found on file at the City of Los Angeles covering the site.

The subject site was previously explored by this firm in 2004 to address a proposed subdivision of the parcel into (11) lots for future residential development. Additional exploration was performed to assess the hazard of fault rupture on the subject site. The overall subsurface exploration consisted of ten (10) trenches excavated using a backhoe and generally encountered fill, colluvium and alluvial fan deposits over bedrock. The exploration logs are provided in Appendix I.

Stratigraphy

The site is underlain by intrusive Cretaceous granitic rock, which is covered mainly by Pleistocene aged fan deposits and artificial fill across the central and eastern portions of the site. Holocene aged fan deposits and artificial fill cover the southwestern portion of the site. Bedrock is exposed in small local outcrops across the northern portion of the site and within the exploration (Trench D) in the northwest portion of the site. No bedrock outcrops were encountered on the southern and eastern portions of the site or within any of the subsurface exploration along Day Street. However, the Cretaceous granitic bedrock is presumed to underlie the alluvial deposits across the site. The earth materials encountered on the subject property are briefly described below. Approximate depths and detailed descriptions of the geologic materials encountered during the field exploration are provided in the Exploration Logs in Appendix I.

Artificial Fill (Af)

Previous grading has resulted in fill placement along the southern portions on the subject site. Aerial photography interpretations indicate that fill materials were emplaced during the grading of the adjacent residential development. It is thought that the subject site may have been a borrow area and stockpile area for grading on the adjacent tract on the southwest side of Day Street. Fill was encountered in all of the trenches excavated in the southern portion of the site near the base of the ascending cut slope, and fill depths range up to about (12) feet in thickness. The fill generally consists of silty sand varying in color from medium brown to gray. Occasional rock fragments that generally range up to about (6) inches in diameter are dispersed throughout the fill. The approximate limit of the existing fill is shown on the enclosed geologic map.

Colluvium (Qc)

Colluvium deposits occur at the toe of most slopes on the project site and are the result of erosion and redeposition of bedrock materials and sediments. These deposits were encountered within trench TP-B and are exposed along the face of the cut slope. The colluvium encountered in the trench generally consists of pale brown (10YR 6/3) silty sand with abundant rock fragments that generally range up to (7) inches in diameter. The colluvium encountered is in a depositional contact with the underlying Older Fan Deposits.

Quaternary Fan Deposits (Qf)

Alluvial fan deposits occupy the western portion of the site, west of trench TP-4. The Holocene aged alluvial fan deposits are weathered bedrock material and sediments that have been eroded from the surrounding steep hillsides and deposited across the toe of slopes, typically where a canyon emerges out of a range front. The fan deposits primarily consists of light gray, moderately dense, silty sand with abundant clasts that generally range between up to (12) inches in diameter. These deposits were encountered within trench TP-5 to the total depth explored.

Older Fan Deposits ($Qof_{1,2,3}$)

The older alluvial fan deposits encountered within the trenches (excluding trenches TP-5 and TP-D) are associated with the Pleistocene-aged deposits of Cook's Fan. At the site, these deposits represent the northwestern extent of Cooks Fan which stems from Cooks Canyon located southeast of the subject site. The Cooks Fan deposit observed on site is thickly bedded and can be further subdivided into three units based on the observed material types and depositional contacts encountered within the trenches. These depositional changes are steeply inclined, and may represent deposition across the western foss of the Cooks Fan. Unit Qof_1 represents the younger chronology of the three units which progress in age to Unit Qof_3.

Unit Qof_1

Unit Qof1 generally consists of fine to coarse grained brown (7.5YR 4/3) silty sand with few subrounded clasts that generally range up to (3) inches in diameter. Locally, larger clasts up to (12) inches in diameter were also encountered. These deposits were encountered within trenches TP-4 and TP-A at the southern portion of the site.

Unit Qof_2

Unit Qof_2 generally consists of yellow brown (10YR 5/4) sandy gravel with a matrix of fine to medium grained sand. Unit Qof_2 is characterized by the abundance of subrounded to rounded clasts that generally are highly weathered and range up to (8) inches in diameter. Locally, larger granitic clasts up to (18) inches in length were also encountered. These deposits were encountered within trenches TP-2, TP-3, TP-4, TP-A and TP-B underlying Unit Qof_1 at the southern portion of the site.

Unit Qof_3

Unit Qof_3 generally consists of dark brown (7.5YR 3/3) silty sand with abundant completely weathered subrounded clasts that generally range up to (6) inches in diameter. Larger granitic clasts encountered within Units Qof_1 and Qof_2 are less common in Unit Qof_3. Unit Qof_3 represents the majority of the older fan deposits exposed on the site and underlies Unit Qof_2. These deposits were encountered within all of the trenches except TP-5 and TP-D.

Wilson Diorite ($wd_{1,2,3}$)

Granitic bedrock was encountered within the trench TP-D and is associated with the Cretaceous-aged Wilson Diorite. The bedrock consists of varying compositions quartz diorite and granodiorite. For this study, the bedrock encountered within the trench TP-D exposure is further divided based on, mineral compositions, degrees of weathering, rock strength and the observed color. However, the bedrock is all considered to be the Cretaceous-aged Wilson Diorite.

Unit wd_1

Unit wd_1 was encountered in the northern portions of trench TP-D (from station 0 to about station 46 feet). Unit wd1 is generally light gray to light yellow brown, massive, fine to medium grained, with weak to moderate rock strength and is intensely weathered.

Unit wd_2

Unit wd2 was encountered across the center of the trench TP-D. Unit wd_2 is generally gray to dark brown, blocky, fine to medium grained, with moderate to strong rock strength and is highly weathered. This unit is locally highly fractured with closely spaced, tight and randomly oriented joints.

Unit wd_3

Unit wd_3 was encountered across the southern portions of trench TP-D and below Unit wd_2. Unit wd_3 is generally light gray to white, massive, fine to medium grained, with weak to moderately strong rock strength and is highly weathered.

Excavation Characteristics

Subsurface exploration was performed through the use of a backhoe excavating into fill, colluvium, alluvial fan deposits, and bedrock. The bedrock encountered during the exploration consists of granitic rock. Extremely hard layers of bedrock were encountered in the explorations. Thus, excavating into the bedrock during construction will be difficult. Typically, the hardness of bedrock increases with depth as the degree of weathering decreases. If hard or well cemented bedrock is encountered, coring or the use of heavy jack hammers may be necessary.

Additionally, excavation difficulty is considered normal within the sediment deposits encountered and should not be limited to consideration of rippability of the earth material. Cohesionless sandy material, although easy to remove, may be subject to sloughing and caving. Therefore difficulty may be encountered maintaining an open excavation. Fine grained materials such as clays and silts may increase in density with depth due to overburden pressure. Thus, difficulty excavating into the material may increase with depth.

Geologic Structure

The local area has been uplifted and intruded by a granitic pluton by past tectonic forces Locally, the onsite bedrock structure is generally massive in the proposed development area and similar to the regional structure. Randomly oriented, discontinuous joints were observed within the bedrock. However, no dominant patterns of adversely orientated fractures or joints were observed during the subsurface investigation.

Critical anticipated bedrock structure is depicted on Geologic Cross Section X, Y, and Z. Preliminary geologic data indicates the proposed development is favorable from the standpoint of geology and geotechnical engineering, provided the recommendations contained herein are followed and maintained.

Landslides

Landslides are a mass wasting phenomenon in mountainous and hillside areas which include a wide range of movements. In Southern California common slope movements include shallow surficial slumps and flows, deep-seated rotational and translational bedrock failures, and rock falls. Landslides occur when the stability of the slopes change to an unstable condition resulting from a number of factors. Common natural factors include the physical and/or chemical weathering of earth materials, unfavorable geologic structure relative to the slope geometry, erosion at the toe of a slope, and precipitation. These factors may be further aggravated by human activities such as excavations, removal of lateral support at the toe of a slope, surcharge at the top of a slope, clearing of vegetation, alteration of drainage, and the addition of water from irrigation and leaking pipes.

Ancient or recent bedrock landslides were not observed on the property. Also, no recent surficial slope failures or slumps were observed within the proposed project area on the property.

Slope Stability

A gross stability analysis (static and pseudostatic) was performed on cross section Y & Z that are considered to represent the most critical profile based on the geologic structure and topography. The analysis was performed using the computer program PCSTABL with the user interface STABL for Windows by Geotechnical Software Solutions, LLC. A summary of the stability analysis is provided below with calculations contained within Appendix III.

Summary of slope stability analyses:

Section	γ_{sat} (pcf)	C (psf)	Phi (deg)	Static Factor of Safety	Pseudo-Static Factor of Safety
Y	145 135	1000 250	35 33	1.82	1.06
Z	145 135	1000 250	35 33	1.77	1.04

The assessment of surficial slope stability was based on the infinite slope with seepage parallel to the slope surface model. The analysis indicates that the slope is surficially stable.

Seismic Hazards

Earthquake Faults

The Alquist-Priolo Earthquake Fault Zoning (AP) Act was passed into law following the destructive February 9, 1971 San Fernando earthquake. The intent of the Act is to increase public safety by reducing the siting of most structures for human occupancy across an active fault. The Act only addresses the hazard of surface fault rupture and is not directed toward other earthquake hazards. The subject site is located within an Alquist-Priolo Earthquake Fault Zone delineated by the State of California. A fault rupture hazard investigation was performed on the subject site to address the proposed development. Active faults were interpreted from the exposures within the seismic trench TP-D along the northwestern portions of the property. These faults are plotted on the attached geologic map enclosed herein. The City of Los Angeles requires a structural setback from the active faults. A detailed discussion of the faulting and structural setbacks is provided in the referenced fault investigation report for the subject development. The structural setbacks for the fault zones are depicted on the geologic map. A general discussion of major faults in Southern California is provided below. The general locations of major faults within Southern California are depicted on a fault map provided by the USGS in Appendix I.

Active Faults

The following active faults are capable of producing seismic waves (ground shaking) on the subject property. A summary description of the closest active faults and potentially active faults to the site are described herein and labeled by number on the map below. An active fault, as defined by the State Mining and Geology Board, is one, which has "had surface displacement within Holocene time (about the last 11,000 years)".

The San Andreas Fault zone (42) is the dominant active fault in California. Geologic studies show that over the past 1,400 to 1,500 years large earthquakes have occurred at about 150-year intervals on the southern San Andreas Fault. It consists of numerous subparallel faults of varied lengths in a zone generally 0.3 to 1.5 km wide in Southern California. The dip of the fault is near vertical and the sense of motion is right-lateral. Historically, the 1857 Fort Tejon earthquake with an estimated magnitude of 7.9 ruptured the ground surface from the vicinity of

Cholame (near Paso Robles) to somewhere between the Cajon Pass and San Gorgonio P (Wrightwood), approximately 200 miles. Studies of offset stream channels indicate that much as (29) feet of movement occurred in 1857. The fault extends from the Gulf of Califo northward to the Cape Mendocino area where it continues along the ocean floor, approxima 750 miles in length.

The Northridge earthquake occurred on January 17, 1994, in the San Fernando Valley. T epicenter was about 1 mile south-southwest of Northridge at a focal depth of 12 miles. T surface wave magnitude was issued by the National Earthquake Information Center at Mw=6 This event occurred on a previously unrecognized south-dipping blind reverse fault with surface rupture. This earthquake produced the strongest ground motions ever instrument recorded in an urban setting in North America. Damage was wide-spread with sections of ma freeways collapsed include some parking structures and office buildings. Common surf disruptions included buckled curbs and sidewalks, fissured concrete and asphalt, and rupture utility lines which are generally aligned in northwest and east-west directions. Shattered rid were reported along Mulholland Drive in the Sherman Oaks area, consisting of intense grou disturbances associated with strong vibratory ground motions within the north trending rid underlain by shale of the Lower Modelo formation.

The Whittier-Elsinore fault zone (60) consists of several subparallel, overlapping and en eche fault strands in a zone up to 1.2 km wide. It extends nearly 125 miles from the Mexican bor to the northern edge of the San Fernando Valley. Seismicity includes the Whittier Narro earthquake of October 1, 1987 with a magnitude of 5.9 and an epicenter in the city Rosemead. This earthquake occurred on a previously unknown and concealed thrust fa There was no reported surface rupture from the earthquake. Also, numerous close a scattered small earthquakes have occurred in historic time near and along the fault.

The San Fernando fault (45) consists of five major en echelon strands at least 9.5 miles length. The "San Fernando" earthquake of February 9, 1971 produced a magnitude of Mw at a depth of 8.4 km along an east west trending reverse fault with a northerly dip. The length the surface rupture was about 9.5 miles and ground shaking lasted for approximately seconds. The earthquake ruptured the northwestern end of the Sierra Madre Fault zo forming the San Fernando Fault. Major damage included the Olive View and Vetera Administration Hospitals and collapse of freeway overpasses. Landslides occurred in the Up Lake area of Van Norman Lakes. Additionally the Van Norman Dam and the Pacoima D were severely damaged.

The eastern portion of the Santa Susana fault (52) ruptured during the 1971 San Fernan Earthquake. The Santa Susana fault consists of several strands in a zone as wide as 1 km generally strikes from north 75 degrees west to north 50 degrees east and dips to the no The fault is a high angle reverse fault. The fault appears to have been generated by northe southwest oriented compressional stress.

The Newport-Inglewood fault zone (31) consists of several strands that extend from offshore Laguna Beach to either merge with or be truncated by the Malibu-Santa Monica fault zone n Beverly Hills. The fault has a length of about 45 miles. It was the source of the "Long Bea earthquake, which occurred on March 10, 1933 with a magnitude of 6.3. Numerous sm earthquakes have occurred in historic time along and near the fault zone. The fault zon easily observed by an alignment of hills and mesas including Cheviot Hills, Baldwin Hills, Rosecrans Hills, Dominguez Hills, Signal Hill, Reservoir Hill, Alamitos Heights, Landing Hill, Bolsa Chica Mesa, and Newport Mesa.

In June 1995, two portions of the Malibu Coast fault zone (27) were reclassified as active fault zones by the State of California. On August 16, 2007, the fault zone near the east side of Malibu Bluff Park was removed from the State of California Earthquake Fault Zone map by the State of California. The east west trending Malibu Coast fault consists of several subparallel strands in a zone as wide as 0.5 km, with a length of at least 17 miles. It strikes east west and dips (45) to (80) degrees to the north. The Malibu Coast fault has the potential to produce a large Maximum Credible Peak and Repeatable Acceleration on the subject property. The duration of the Malibu Coast fault is estimated at (11) seconds assuming fault end nucleation and unidirectional rupture propagation, (Bolt, 1981). The Malibu Coast fault is thought to be part of other faults such as the Santa Monica fault and Hollywood fault that separate the Transverse Ranges on the north from the Peninsula Range on the south. Two Malibu Earthquakes occurred with Magnitudes of M_L 5.2 and M_L 5.0 on January 1, 1979 and January 18, 1989, respectively. It was reported that only minor damage occurred in the areas closest to the epicenter.

The Hollywood fault zone (22) extends along the base of the Santa Monica Mountains. This fault was added to the list of active fault by the State of California in 2014. Generally, the Hollywood fault extends eastward for a distance of 15 km through Beverly Hills, West Hollywood, and Hollywood to the Los Angeles River. The fault is primarily expressed at the ground surface by scarp-like features. This is a left–reverse fault with an estimated slip rate between 0.33 mm/yr and 0.75 mm/yr, (Petersen and Wesnousky 1994).

The Raymond fault (39) is a combination fault with reverse and left slip movement that acts as a groundwater barrier within the densely populated San Gabriel Valley. The activity of the fault is attested to by the numerous geomorphic features found along its entire length of approximately 14 miles. Scattered small earthquakes have occurred north of the fault trace. It may be the source of the 1855 Los Angeles earthquake. The Raymond fault is an east-trending fault made up of other faults such as the Hollywood and Santa Monica faults that separate the Transverse Ranges on the north form the Peninsula Range on the south.

The Sierra Madre fault zone (53) is often divided into five main segments; Vasquez Creek fault, Clamshell fault (10), Sawpit Canyon fault (10), Duarte fault and the Cucamonga fault (14). The Sierra Madre earthquake of June 28, 1991 (Mw5.8) was in the San Gabriel Mountains. An estimated 33.5 million dollars of damage has been reported. The Sierra Madre fault zone is about 75 km long. It's a thrust fault system along the south edge of the San Gabriel Mountains. The east end of the Sierra Madre fault zone intersects the San Jacinto fault and the San Andreas Fault. The 1971 San Fernando earthquake occurred on the San Fernando-Sunland segment of the Sierra Madre fault zone.

The San Gabriel fault (46) consists of several en echelon fault strands in a zone approximately 0.5 km wide, with a length of about 90 miles. The fault trends northwestward and subparallel to the San Andreas Fault. As of March 1, 1988, a portion of the Newhall segment of the fault zone was reclassified as an active fault. Fault activity has been dated between 1550 and 3500 years before present within the Newhall segment. The youngest ground rupture event has broken alluvial beds to within five feet of the ground surface. Geologic evidence suggests 38 miles of right lateral offset has occurred between 14 million and 3 million years ago and may h functioned as an ancestral branch of the San Andreas Fault. Recent studies suggest that major strike slip movement has become inactive and dip slip movement is active at the pre time.

Potentially Active Faults

A potentially active fault, as defined by the State Mining and Geology Board, is one, which had surface displacement during Quaternary time (last 1.6 million years). "These faults those based on available data along which no known historical ground surface rupture earthquakes have occurred. These faults, however, show strong indications of geologi recent activity". The following list provides potentially active faults that are capable of produ seismic waves (ground shaking) on the property.

The Santa Monica fault (50) extends east from the coastline in Pacific Palisades through S Monica and West Los Angeles and merges with the Hollywood fault. Several local geolo believe portions of the Santa Monica fault zone are active. Currently, it is listed by the Sta California as a potentially active fault. Portions of the fault zone may change to "active" an placed within the Alquist-Priolo Earthquake Fault Zone as additional geologic reports submitted to the State containing evidence of Holocene activity. The Santa Monica fault con of one or more fault strands, with a poorly known geometry. Generally, the fault st northeast 60 to 80 degrees and dips 45 to 65 degrees northwest at depth with a few vertical surface traces. The length of the fault is at least 25 miles. The composite mechanism of fault displacement is a reverse left lateral along the Santa Monica-Hollyw Raymond fault zone. The Santa Monica and Hollywood faults may be part of a larger system that includes Malibu Coast, Raymond and Cucamonga fault system. This fault forms the central portion of a major tectonic boundary separating the east west tre Transverse Ranges province to the north from the northwest trending Peninsular Ra province to the south.

The Benedict Canyon fault zone trends eastward through the Santa Monica Mountains. fault may be part of the Hollywood-Santa Monica-Raymond fault system. The activity c fault is based on offsets in groundwater bearing sediments that correlate with steep di gravity gradients. The fault extends through Universal City and along the north side c eastern part of the Santa Monica Mountains.

The Simi fault (54) consists of a single strand that bifurcates at the western end. Gener strikes north 70-80 degrees east and dips 60 to 75 degrees north with a length of about 31-

The Mission Hills fault (30) is an east west trending fault with a length of about 9 km. Th is presumed to be a single strand that strikes north 80 degrees east to east west and dips 80 degrees to the north.

The Chatsworth fault (8) is a reverse fault which juxtaposes Cretaceous Chatsworth forr and Paleocene Martinez formation over Miocene Modelo formation within the San Fer Valley.

The Palos Verdes Hills fault (35) consists of several en echelon strands locally in a zone as wide as 2 km with a length of 50 miles. It strikes north between 20 and 60 degrees west with dips of 70 degrees to the southwest.

Seismic Effects

During an earthquake there are several primary geologic hazards such as ground rupture, ground shaking, landslides, and liquefaction that can adversely affect property, structures, and improvements. On hillside properties, the potential exists for landsliding from ground shaking which may adversely affect property, structures, and improvements. The State of California has prepared maps that detail areas which may require assessment for ground rupture, landsliding and/or liquefaction. Strong ground shaking is the primary hazard that causes damage from earthquakes and these areas have been zoned with a high level of seismic shaking hazard. The historical earthquake record in Southern California is less than 200 years; therefore, potential damage from a seismic event is not limited areas that have experienced damage in the past. Based on the above discussion, earthquake insurance with building code upgrades is suggested.

There are several active and/or potentially active faults that could possibly affect the site within Los Angeles County. The site is located within Seismic Zone 4. Although all of Southern California is within a seismically active region, some areas have a higher potential for seismic damage than others. The current scientific technology does not provide for accurate prediction of the time, location, or magnitude of an earthquake event.

It should be understood that the following discussion is an evaluation of risk and degree of potential damage to a structure if a fault were to rupture on or near the site and does not imply that a fault may or may not be present beneath the site. An assessment of damage to the structure is based on the Modified Mercalli Intensity Scale which is correlated to observed damage from seismic events. Intensity/damage associated with an earthquake is not directly correlated to magnitude. For a given magnitude of an earthquake, the intensity/damage to a structure may vary depending on the subsurface earth materials, type of fault rupture, hypocenter depth, and local building practices in effect during the construction of a structure.

An evaluation of the seismic effects on a property is designed to provide the client with rational and believable seismic data that could affect the property during the lifetime of the proposed improvements. The minimum design acceleration for a project is listed in the Building Code. It is recommended that the structural design of the proposed project be based on current design and acceleration practices of similar projects in the area. The project structural designer should review and verify all of the seismic design parameters prior to utilizing the information for the design.

Ground Rupture

Ground rupture is the result of movement from an active fault. A fault is a fracture in the crust of the earth along which rocks on one side have moved relative to those on the other side. Faulting was encountered within the seismic trench TP-D which is located at the northwest portion of the site. The encountered fault trends are shown on the enclosed Geologic Map. The potential trends of the encountered faults were further evaluated based on the site topography and geomorphology. Based on the referenced fault rupture hazard investigation, no known active fault is mapped within the proposed development area along Day Street.

Ground Shaking

Ground shaking caused by an earthquake is likely to occur at the site during the lifetime of the development due to the proximity of several active and potentially active faults. Generally, on a regional scale, quantitative predictions of ground motion values are linked to peak acceleration and repeatable acceleration, which are a response to earthquake magnitudes relative to the fault distance from the subject property. Southern California major earthquakes are generally the result of large-scale earth processes in which the Pacific plate slides northwestward relative to the North American plate at about 2 inches/year.

The potential for lurching, surface manifestations, landslides, and topographic related features from ground/seismic shaking can occur almost anywhere in Southern California. Proper maintenance of properties can mitigate some of the potential for these types of manifestations but the potential cannot be completely eliminated. Many structures were built before earthquake codes were adopted; others were built according to codes formulated when less was known about the intensity of near-fault shaking. Therefore, the margin of safety is difficult to quantify.

A publicly available computer program provided by the United States Geological Survey (USGS was utilized for the probabilistic prediction of peak horizontal ground acceleration from digitize design maps of Maximum Considered Earthquake (MCE) ground response. A summary of th seismic design parameters is provided in Appendix III. The project structural designer shoul verify all of the input parameters and review all of the resulting seismic design parameters pric to utilizing the information for the design.

Earthquake Induced Landslides

The State of California has prepared Seismic Hazard Zone Reports to regionally map areas c potential increased risk of permanent ground displacement based on historic occurrence c landslide movement, local topographic expression, and geological and geotechnical subsurfac conditions. The maps may not identify all areas that have potential for earthquake-induce landsliding, strong ground shaking, or other earthquake-related geologic hazards. The subje site is located within an earthquake-induced landslide hazard zone on the State of Californi Seismic Hazard Map.

Ancient or recent bedrock landslides were not observed on the property. Also, no rece surficial slope failures or slumps were observed within the proposed project area on th property.

Based on the stability analysis earthquake-induced landslide hazard at the subject site is low.

Liquefaction

The State of California has prepared Seismic Hazard Zone Reports to regionally map are where historic occurrence of liquefaction, or local geological, geotechnical and groundwat conditions indicate a potential for permanent ground displacement. The maps may not identify all areas that have potential for liquefaction, strong ground shaking, and other earthquake and geologic hazards. The subject site is not located within a liquefaction hazard zone on the State of California Seismic Hazard Zone Map.

Liquefaction is a process by which sediments below the water table temporarily lose strength and behave as a viscous liquid rather than a solid. The types of sediments most susceptible are clay-free deposits of sand and silts; occasionally gravel liquefies. Liquefaction can occur when seismic waves, primarily shear waves, pass through saturated granular layers distorting the granular structure, and causing loosely packed groups of particles to collapse. These collapses increase the pore-water pressure between grains if drainage cannot occur. If the pore-water pressure rises to a level approaching the weight of the overlying soil, the granular layer temporarily behaves as a viscous liquid rather than a solid.

In the liquefied condition, soil may deform with little shear resistance; deformations large enough to cause damage to buildings and other structures are called ground failures. The ease with which a soil can be liquefied depends primarily on the looseness of the material, the depth, thickness and areal extent of the liquefied layer, the ground slope and the distribution of loads applied by buildings and other structures.

Liquefaction induced ground deformations (detailed below) will have an effect on the proposed and existing development that can result in significant structural damage, collapse or partial collapse of a structure, especially if there is significant differential settlement or lateral spreading between adjacent structural elements. Even without collapse, significant settlement or lateral spreading could result in significant structural damage including, but not limited to, blocked doors and windows that could trap occupants.

A detailed subsurface analysis can be performed to determine the liquefaction potential on the subject site and provide recommendations to mitigate the effects of liquefaction. A proposal for a detailed analysis will be prepared if requested.

Surface Manifestations

The determination of whether surface manifestation of liquefaction (such as sand boils, ground fissures etc.) will occur during earthquake shaking at a level-ground site can be made using the method outlined by Ishihara (1985). It is emphasized that settlement may occur, even with the absence of surface manifestation. Youd and Garris (1994 and 1995) evaluated the Ishihara method and concluded that the method is not appropriate for level ground sites subject to lateral spreading and/or ground oscillation.

Lateral Spreads

Whereas the potential for flow slides may exist at a building site, the degradation in undrained shear resistance arising from liquefaction may lead to limited lateral spreads (of the order of feet or less) induced by earthquake inertial loading. Such spreads can occur on gently sloping ground or where nearby drainage or stream channels can lead to static shear stress biases on essentially horizontal ground (Youd, 1995). At larger cyclic shear strains, the effects of dilation may significantly increase post liquefaction undrained shear resistance. However, incremental permanent deformations will still accumulate during portions of the earthquake load cycles when low residual resistance is available. Such low resistance will continue even while large permanent shear deformations accumulate through a ratcheting effect. Such effects have recently been demonstrated in centrifuge tests to study liquefaction induced lateral spreads, as described by Balakrishnan et al. (1998). Once earthquake loading has ceased, the effects of dilation under static loading can mitigate the potential for a flow slide.

It is clear from past earthquakes that damage to structures can be severe, if permanent ground displacements on the order of several feet occur. However, during the Northridge earthquake significant damage to building structures (floor slab and wall cracks) occurred with less than one (1) foot of lateral spread. The complexities of post-liquefaction behavior of soils noted above, coupled with the additional complexities of potential pore water pressure redistribution effects and the nature of earthquake loading on the sliding mass, lead to difficulties in providing specific guidelines for lateral spread evaluations.

Seismically Induced Settlements

Seismic settlement occurs when cohesionless soils densify as result of ground shaking. Typically seismically induced settlement is greatest in loose cohesionless sands. Lee and Albaisa (1974) and Yoshimi (1975) studied the volumetric strains (or settlements) in saturated sands due to dissipation of excess pore pressures generated in saturated granular soils by the cyclic ground motions. The volumetric strain, in the absence of lateral flow or spreading, results in settlement. Liquefaction-induced settlement could result in collapse or partial collapse of a structure, especially if there is significant differential settlement between adjacent structural elements. Even without collapse, significant settlement could result in blocked doors and windows that could trap occupants.

CONCLUSIONS

1. Based on the results of this investigation and a thorough review of the proposed development, as discussed, the project is suitable for the intended use providing the following recommendations are incorporated into the design and subsequent construction of the project. Also, the development must be performed in an acceptable manner conforming to building code requirements of the controlling governing agency.

2. Based on the State of California Seismic Hazard Maps, the subject site is not located within a liquefaction hazard zone.

3. Based on the State of California Seismic Hazard Maps, the subject site is located in an earthquake-induced landslide hazard zone.

4. The SITE CLASS based on California Building Code is D.

5. Based upon field observations, laboratory testing and analysis, the alluvial fan deposits found in the exploration trenches should possess sufficient strength to support the recommended compacted fill and proposed development.

RECOMMENDATIONS

Specific

1. To create a uniform building pad for the proposed development, the existing fill and upper (5) feet of fan deposits should be removed replaced as compacted fill. In addition, the proposed removals should extend a minimum of three feet below the proposed foundations. Removal and recompaction of the exploration trench backfill will be required.
2. The proposed residences should be supported on foundations embedded into certified fill.
3. The property owner shall maintain the site as outlined in the Drainage and Maintenance Section.

Building Setbacks

The construction of buildings and structures on or adjacent to slopes steeper than 3:1 (horizontal to vertical) in gradient shall be setback from the slopes in accordance with the requirements of the applicable governmental agency.

In general, all foundations on or adjacent to a descending slope shall be located a distance of one-third of the vertical height of the slope (H/3) to provide vertical and lateral support for the foundation. This distance is measured horizontally from the face of the foundation to the face of the bearing material. This horizontal distance does not need to exceed (40) feet. Where the slope is steeper than 1:1 (horizontal to vertical), the required setback shall be measured from an imaginary plane at (45) degrees to the horizontal, projected upward from the toe of the slope.

In general, buildings and structures on or adjacent to an ascending slope shall be located a distance of one-half of the vertical height of the slope (H/2) to provide sufficient protection from slope drainage, erosion, and shallow failures. This distance is measured horizontally from the face of the building/structure to the toe of the slope. This horizontal distance does not need to exceed (15) feet. Where the slope is steeper than 1:1 (horizontal to vertical), the toe is considered to be at the intersection of a horizontal plane from the top of the foundation and an imaginary plane tangent to the slope at (45) degrees to the horizontal.

Drainage and Maintenance

Maintenance of properties must be performed to minimize the chance of serious damage and/or instability to improvements. Most problems are associated with or triggered by water. Therefore, a comprehensive drainage system should be designed and incorporated into the final plans. In addition, pad areas should be maintained and planted in a way that will allow this drainage system to function as intended. The property owner shall be fully responsible for dampness or water accumulation caused by alteration in grading, irrigation or installation of improper drainage system, and failure to maintain drain systems. The following are specific drainage, maintenance, and landscaping recommendations. Reductions in these recommendations will reduce their effectiveness and may lead to damage and/or instability to the improvements. It is the responsibility of the property owner to ensure that improvements, structures and drainage devices are maintained in accordance with the following recommendations and the requirements of all applicable government agencies.

Drainage

Positive pad drainage should be incorporated into the final plans. The pad should slope away from the footings at a minimum five percent slope for a horizontal distance of ten feet. In areas where there is insufficient space for the recommended ten foot horizontal distance concrete or other impermeable surface should be provided for a minimum of three feet adjacent the structure. Pad drainage should be at a minimum of two percent slope where water flow over lawn or other planted areas. Drainage swales should be provided with area drains about every fifteen feet. Area drains should be provided in the rear and side yards to collect drainage. All drainage from the pad should be directed so that water does not pond adjacent to the foundations or flow toward them. Roof gutters and downspouts are required for the proposed structures and should be connected into a buried area drain system. All drainage from the site should be collected and directed via non-erosive devices to a location approved by the building official. Area drains, subdrains, weep holes, roof gutters and downspouts should be inspected periodically to ensure that they are not clogged with debris or damaged. If they are clogged or damaged, they should be cleaned out or repaired.

Landscaping (Planting)

The property owner is advised not to develop planter areas between patios, sidewalk and structures. Planters placed immediately adjacent to the structures are not recommended. If planters are proposed immediately adjacent to structures, impervious above-grade or below-grade planter boxes with solid bottoms and drainage pipes away from the structure are suggested. All slopes should be maintained with a dense growth of plants, ground-covering vegetation, shrubs and trees that possess dense, deep root structures and require a minimum o irrigation. Plants surrounding the development should be of a variety that requires a minimum o watering. It is recommended that a landscape architect be consulted regarding plantin adjacent to improvements. It will be the responsibility of the property owner to maintain the planting. Alterations of planting schemes should be reviewed by the landscape architect.

Irrigation

An adequate irrigation system is required to sustain landscaping. Over-watering resulting i runoff and/or ground saturation must be avoided. Irrigation systems must be adjusted t account for natural rainfall conditions. Any leaks or defective sprinklers must be repaire immediately. To mitigate erosion and saturation, automatic sprinkling systems must be adjuste for rainy seasons. A landscape architect should be consulted to determine the best times f landscape watering and the proper usage.

Pools/Plumbing

Leakage from a swimming pool or plumbing can produce a perched groundwater condition th may cause instability or damage to improvements. Therefore, all plumbing should be leak-free

Grading and Earthwork

Proposed grading will consist of removal and recompaction in the area of the proposed residences and conventional cut and fill grading observations for the proposed debris basins.

Remedial grading is recommended within the building areas in order to remove the existing fill, upper portion of the alluvial soils. Based on the conditions encountered in the explorations the recommended removals are anticipated to depths of about five feet from the existing grade. Deeper removals will be required to remove the exploration trench backfill and replace as compacted fill. The over-excavation should extend a minimum of five feet beyond the building perimeters, and to an extent equal to the depth of fill below the new foundations. If the proposed structure incorporates exterior columns (such as for an overhang) the over-excavation should also encompass these areas.

Following the completion of the over-excavation, the subgrade soils should be evaluated by the project geotechnical engineer to verify their suitability to support the structural fill as well as to support the foundation loads of the proposed development. This evaluation may include probing and proof-rolling to identify any soft, loose or otherwise unstable soils that must be removed. Some localized areas of deeper excavation may be required if additional fill materials or dry, loose, porous or otherwise unsuitable materials are encountered at the base of the over-excavation.

Flatland Grading

1. Prior to commencement of work, a pre-grading meeting shall be held. Participants at this meeting will consist of the contractor, the owner or his representative, and the soils engineer. The purpose of the meeting is to avoid misunderstanding of the recommendations set forth in this report that might cause delays in the project.

2. Prior to placement of fill, all vegetation, rubbish, and other deleterious material should be disposed of offsite. The proposed structures should be staked out in the field by a surveyor. This staking should, as a minimum, include areas for overexcavation, toes of slopes, tops of cuts, setbacks, and easements. All staking shall be offset from the proposed grading area at least five feet (5'). Line and grade verification is not provided by GeoConcepts, Inc.

3. The natural ground, that is determined to be satisfactory for the support of the filled ground, shall then be scarified to a depth of at least six inches (6") and moistened as required. The scarified ground should be compacted to at least 90 percent of the maximum laboratory density (ASTM D 1557).

4. The fill soils shall consist of materials approved by the project Soils Engineer or his representative. These materials may be obtained from the excavation areas and any other approved sources, and by blending soils from one or more sources. The material used shall be free from organic vegetable matter and other deleterious substances, and shall not contain rocks greater than eight inches (8") in diameter nor of a quantity sufficient to make compaction difficult.

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5. The approved fill material shall be placed in approximately level layers six inches (6") thick, and moistened as required. Each layer shall be thoroughly mixed to attain uniformity of moisture in each layer.

   When the moisture content of the fill is (3) percent or more below the optimum moisture content, as specified by the Soils Engineer, water shall be added and thoroughly mixed in until the moisture content is within (3) percent of the optimum moisture content.

   When the moisture content of the fill is (3) percent or more above the optimum moisture content as specified by the Soils Engineer, the fill material shall be aerated by scarifying or shall be blended with additional materials and thoroughly mixed until the moisture content is within (3) percent or less of the optimum moisture content.

   Each layer of fill material shall be compacted to a minimum of (90) percent of the maximum dry density as determined by ASTM D 1557, using approved compaction equipment. Where cohesionless soil having less than (15) percent finer than (0.005) millimeters is used for fill, the fill material shall be compacted to a minimum of (95) percent of the maximum dry density.

6. Review of the fill placement should be provided by the Soils Engineer or his representative during the progress of grading. In general, density tests (ASTM D 1556) and (*ASTM D 2922 & 3017*) will be made at intervals not exceeding two feet (2') of fill height or every 500 cubi yards of fill placed.

7. During the inclement part of the year, or during periods when rain is threatening, all fill that ha been spread and awaits compaction shall be compacted before stopping work for the day o before stopping because of inclement weather. These fills, once compacted, shall have th surfaces sloped to drain to one area where water may be removed.

   Work may start again, after the rainy period, once the site has been reviewed by the Soi Engineer and he has given his authorization to resume. Loose materials not compacted pri to the rain shall be removed and aerated so that the moisture content of these fills will b within (3) percent of the optimum moisture content.

   Surface materials previously compacted before the rain, shall be scarified, brought to th proper moisture content, and re-compacted prior to placing additional fill, if deemed necessa by the Soils Engineer.

8. Review of geotechnical data available for the local vicinity of the site indicates that sep tanks, seepage pits, or leach fields may be encountered during site grading. If encountere these should be drained of effluent or drilled out if they have been backfilled. The cleaned- area should be inspected by the soils engineer and governing inspector prior to backfill. T pool may be filled with approved compacted fill, lean concrete, or gravel. Whichever bac material is selected, at least five feet (5') of approved manmade fill, placed at 90 perc relative compaction should cap the pool.

Foundations

It is recommended that the proposed structure be founded into certified compacted fill. All foundations shall maintain the required code setback from any slope.

The minimum continuous footing size is (12) inches wide and (24) inches deep into the compacted fill, measured from the lowest adjacent grade. Continuous footings may be proportioned, using a bearing value of (2000) pounds per square foot. Column footings placed into the compacted fill may be proportioned, using a bearing value of (2500) pounds per square foot, and should be a minimum of (2) feet in width and (24) inches deep, below the lowest adjacent grade.

All continuous footings shall be reinforced with a minimum of (4) #(5) bars, two placed near the top and two near the bottom. Reinforcing recommendations are minimums and may be revised by the structural engineer.

The bearing values given above are net bearing values; the weight of concrete below grade may be neglected. These bearing values may be increased by one-third (1/3) for temporary loads, such as, wind and seismic forces.

All footing excavation depths will be measured from the lowest adjacent grade of recommended bearing material. Footing depths will not be measured from any proposed elevations or grades. Any foundation excavations that are not the recommended depth <u>into</u> the recommended bearing materials will not be acceptable to this office.

Lateral loads may be resisted by friction at the base of the conventional foundations and by passive resistance within the compacted fill. A coefficient of friction of (0/4) may be used between the foundations and the compacted fill. The passive resistance may be assumed to act as a fluid with a density of (300) pounds per cubic foot. A maximum passive earth pressure of (3000) pounds per square foot may be assumed.

Settlement

Settlement of the proposed residences will occur. Settlement of (1/8) to (1/4) inches between walls, within 20 feet or less, of each other, and under similar loading conditions, are considered normal. Total settlement on the order of (1/2) inches should be anticipated.

Expansive Soils

Expansive soils were not encountered on the subject property. Expansive soils can be a problem, as variation in moisture content will cause a volume change in the soil. Expansive soils heave when moisture is introduced and contract as they dry. During inclement weather and/or excessive landscape watering, moisture infiltrates the soil and causes the soil to heave (expansion). When drying occurs the soils will shrink (contraction).

Repeated cycles of expansion and contraction of soils can cause pavement, concrete slabs or grade and foundations to crack. This movement can also result in misalignment of doors and windows. To reduce the effect of expansive soils, foundation systems are usually deepene and/or provided with additional reinforcement design by the structural engineer. Planning c yard improvements should take into consideration maintaining uniform moisture condition around structures. Soils should be kept moist, but water should not be allowed to pond. Thes designs are intended to reduce, but will not eliminate deflection and cracking and do n guarantee or warrant that cracking will not occur.

Excavations

Excavations ranging in vertical height up to (10) feet will be required for the proposed gradin Conventional excavation equipment may be used to make these excavations. Excavatio should expose fill and/or colluvium over fan deposits. These soils are suitable for vertic excavations up to five (5) feet, portions of the excavation above five feet should be trimm back at a 1:1 (h:v) slope gradient. This should be verified by the project geotechnical engine during construction so that modifications can be made if variations in the soil occur.

All excavations should be stabilized within 30 days of initial excavation. If this time is exceede the project geotechnical engineer must be notified, and modifications, such as shoring or slo trimming may be required. Water should not be allowed to pond on top of the excavation, nor flow toward it. All excavations should be protected from inclement weather. This is required keep the surface of the open excavation from becoming saturated during rainfall. Saturation the excavation may result in a relaxation of the soils which may result in failures. Excavatic should be kept moist, not saturated, to reduce the potential for raveling and sloughing dur construction. No vehicular surcharge should be allowed within three feet (3') of the top of cut.

Excavations Maintenance – Erosion Control

The following recommendations should be considered a part of the excavation/erosion cor plan for the subject site and are intended to supplement, but not supersede nor limit the eros control plans produced by the Project Civil Engineer and/or Qualified SWPPP Develo These recommendations should be implemented during periods required by the Building C (typically between the months of October and April) or at any time of the year prior t predicted rain event. Consideration should also be given to potential local sources water/runoff such as existing drainage pipes or irrigation systems that remain in operation du construction activities.

Open Excavations:

All open excavations shall be protected from inclement weather, including areas above ar the toe of the excavation. This is required to keep the excavations from becoming satur the toe of the excavation. Saturation of the excavation may result in a relaxation of the soils which may result in fail Water/runoff should be diverted away from the excavation and not be allowed to flow ove excavation in a concentrated manner.

Hillside Excavations:

All hillside excavations shall be protected during inclement weather and should extend beyond the edges of the excavations in all directions. Plastic sheeting along with stakes, ropes and sandbags may be used to provide protection of the excavations. Water/runoff should be diverted away from the excavation and not be allowed to flow over the excavation.

The project Civil Engineer should provide a plan depicting the required limits of erosion control. Slopes around an open excavation should be trimmed to slope away from the open excavation so that water/runoff will not drain into the excavation. Any trees or planters that might cause failures around an open excavation shall be anchored safely. After the inclement weather has ceased, the excavations shall be reviewed by the project geotechnical engineer and geologist for safety prior to recommencement of work.

Open Trenches/Foundation Excavations:

No water should be allowed to pond adjacent to or flow into open trenches. All open trenches shall be covered with plastic sheeting that is anchored with sandbags. Areas around the trenches should be sloped away from the trenches to prevent water runoff from flowing into or ponding adjacent to the trenches.

After the inclement weather has ceased, the excavations shall be reviewed by the project geotechnical engineer and geologist for safety prior to recommencement of work. Foundation excavations that remain open during inclement weather shall be reviewed by the project geotechnical engineer and geologist prior to the placement of steel and concrete to ensure that proper embedment and contact with the bearing material have been maintained.

Open Pile/Caisson Excavations:

All pile/caisson excavations should be reviewed and poured prior to the onset of inclement weather. It is not recommended that any pile/caisson excavations remain open through any inclement weather. However, if it is necessary to leave pile/caisson excavations open during inclement weather, all water and runoff shall be diverted away from and prevented from entering the pile/caisson excavations. Pile/caisson excavations that remain open during inclement weather shall be reviewed by the project geotechnical engineer and geologist prior to the placement of steel and concrete to ensure that proper embedment has been maintained. The base of all end-bearing caissons shall be re-cleaned to ensure contact with the proper bearing material. All stockpiled cuttings from the pile borings shall be removed.

Grading In Progress:

During the inclement time of the year, or during periods prior to the onset of rain, all fill that has been spread and is awaiting compaction shall be compacted before stopping work for the day or before stopping work because of inclement weather. These fills, once compacted, shall have the surface sloped to drain to one area where water may be removed.

Additionally, it is suggested that all stock-piled fill materials be covered with plastic sheeting. This action will reduce the potential for the moisture content of the fill from becoming too high for compaction. If the fill stockpile is not covered during inclement weather, then aerating the fill reduce the moisture content would be required. This action is generally very time consumi and may result in construction delays.

Work may recommence, after the rain event, once the site has been reviewed by the proje geotechnical engineer.

Retaining Walls

Cantilever retaining walls should be designed to resist an active earth pressure such as th exerted by compacted backfill. Retaining walls up to (10) feet in height may be designed per t following table. The 'active' pressure assumes that the wall will be allowed to deflect 0.01H 0.02H. Basement walls and other walls where horizontal movement is restricted at the top not allowed to deflect shall be designed for at-rest pressure (currently these walls are n proposed).

Surface Slope of Retained Material Horizontal to Vertical	Active Equivalent Fluid Weight p.c.f.
Level	30
5 to 1	32
4 to 1	35
3 to 1	38
2 to 1	43
1½ to 1	55
1 to 1	80

In addition to lateral earth pressure, these retaining walls should be designed to resist t surcharge imposed by the proposed structures, footings, any adjacent buildings, or by adjac traffic surcharge, per the attached figures 11 and 12 obtained from the Naval Facilit Engineering Command, Design Manual 7.02 (Foundation and Earth Structures, pages 74 a 75).

The wall pressure stated assumes that the wall has been backfilled as outlined below wit permanent drainage system. Proper compaction of the backfill is recommended to prov lateral support to adjacent properties. Even with proper compaction of required back settlement of the backfill may occur. Accordingly, utility lines, footings, slabs, or falsew should be planned and designed to accommodate potential settlement.

Walls to be backfilled must be reviewed by the project Geotechnical Engineer prior commencement of the backfilling operation.

1. Adequate permanent drainage is required behind the wall to minimize the buildup hydrostatic pressures. A perforated pipe, with perforations placed down, shall be installed

the base of the wall footing. The pipe shall be encased in at least one foot (1') of three-quarter inch (3/4") gravel. The pipe shall exit from behind the retaining wall and drain to a location approved by the architect or civil engineer.

When space does not permit the installation of standard pipe and gravel drainage system, i.e. walls adjacent the property line, a flat drainage product is acceptable subject to approval of the governing agency. It is recommended that a drainage composite geotextile (such as MiraDrain / QuickDrain) be placed at the base of the proposed retaining wall. The drainage composite geotextile will provide comparable drainage to the conventional four inch perforated pipe encased in gravel per Code Sections 1805.4.2 and 1805.4.3

If a drainage system is not provided the walls should be designed to resist an external hydrostatic pressure due to water in addition to the lateral earth pressure in Retaining Wall section. The entire wall should be design for full hydrostatic pressure based on a water level at the ground surface. In addition, floors would need to be designed for hydrostatic uplift and waterproofed.

2. A continuous vertical drain, consisting of a gravel blanket six inches (6") thick or geotextile vertical drainage system, shall be placed along the back side of the wall to within 2 feet of the ground surface.

3. Water and moisture affecting retaining walls is a common post-construction complaint. Poorly applied or omitted waterproofing can lead to standing water inside the building or efflorescence on the wall.

   It is recommended that the retaining walls be waterproofed. Waterproofing design and inspection of installation is not the responsibility of the geotechnical engineer. GeoConcepts, Inc. does not practice in the field of water and moisture vapor transmission evaluation/mitigation. Therefore, we recommend that a qualified person/firm be engaged/consulted to evaluate the general and specific water and moisture vapor transmission paths and any impact on the proposed development. This person/firm should provide recommendations for mitigation of potential adverse impact of water and moisture vapor transmission on various components of the structure as deemed necessary. The actual waterproofing design shall be provided by the architect, structural engineer or contractor with experience in waterproofing.

4. After the wall backdrain system has been placed and the waterproofing installed, fill may be placed, if sufficient room allows, in layers not exceeding four inches (4") in thickness and compacted to 90 percent of the maximum density, as determined by ASTM D 1557. Where cohesionless soil having less than (15) percent finer than (0.005) millimeters is used for fill, the fill material shall be compacted to a minimum of (95) percent of the maximum dry density.

5. Where space does not permit compaction of material behind the wall (<24 inches wide), a granular backfill shall be used. This granular backfill shall consist of one-half inch (1/2") to three-quarter inch (3/4") crushed rock and should be densified by tamping into place. The crushed rock backfill should not exceed a depth of ten feet.

6. All granular free-draining wall backfills shall be capped with a clayey compacted soil within the upper two feet (2') of the wall backfill. This compacted material should start below the required wall freeboard.

7. A concrete-lined swale drain should be placed behind any retaining wall that can intercept

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surface runoff from upslope areas. This surface runoff shall be transferred to an are approved by the building official.

8. A minimum freeboard of two (2) feet shall be maintained at all times. Any slough, debris c trash should be removed immediately. Swales shall be maintained, by sealing any and a cracks or repairing breaks that occur over the life of the swale.

Lateral Earth Pressure Due to Earth Motion

Cantilever retaining walls should be designed to resist an active earth pressure due to ear motion, if required by the building official, distributed as a triangle pressure. Retaining walls to (10) feet in height may be designed per the following table. The seismic equivalent flu pressure is in addition to static earth pressures.

The seismic loading is based on a horizontal acceleration coefficient of 0.3.

Surface Slope of Retained Material Horizontal to Vertical	Seismically Induced Earth Pressure - Equivalent Fluid Weight p.c.f.
Level	10
5 to 1	13
4 to 1	16
3 to 1	19
2 to 1	22
1½ to 1	25
1 to 1	30

Raised Floors

Raised floor type construction typically results in a lowered grade beneath the residence rel to the exterior grade. The lowered grade often leads to moisture problems under the reside Surface water/moisture can seep through or migrate beneath footings and pond beneat residence. The larger the grade differential between the exterior and interior the more moisture can seep beneath the residence. Soils with clay or silt are most commonly assoc with this type of problem. Prolonged moisture under the residence can lead to growth of fu rotting of wood framing elements and/or mold growth.

To minimize the potential of water/moisture seeps under the residence the following mea are recommended, such as, but not limited to positive drainage away from founda waterproofing the foundations, sealing utility line penetrations through the founda compaction of trench backfill placement, foundation drains and planter drains. Subdrains directly adjacent the footing stemwalls are beneficial but will generally not completely eli water/moisture seeps under the residence. Planter drains which are located away fro footings and extend deeper than the footings are generally more effective. Other metho also be employed such as placement of a vapor barrier and lightly reinforced concrete slab over the earth in the lowered grade areas. The slab should be sloped to drain to area drains.

Adequate ventilation of the subfloor area is critical in preventing high under floor moisture conditions. Consideration should be given to providing more than the minimum Code-required amount of vent space. Larger homes may require mechanical ventilation.

Irrigation for landscaping around the residence can be a contributing factor to moisture intrusion beneath the residence, especially where drainage and/or ventilation is insufficient. In addition to proper drainage and ventilation, the maintenance and proper use of irrigation systems should also be considered to avoid over-irrigation, which may result in runoff and/or ground saturation. Irrigation systems should be adjusted to account for seasonal rainfall conditions and inclement weather. Drought resistant landscaping should be considered to minimize water usage. Damaged or defective irrigation systems should be repaired or replaced to avoid concentrated runoff and/or ground saturation. A landscape professional should be consulted to determine the best landscaping and proper irrigation for your site.

Slabs on Grade

Slabs on grade should be reinforced with minimum #4 reinforcing bars, placed at (16) inches on center each way and supported on compacted fill. Provisions for cracks should be incorporated into the design and construction of the foundation system, slabs, and proposed floor coverings. Concrete slabs should have sufficient control joints spaced at a maximum of approximately 8 feet. Two-car garage slabs should be quartered or saw cut slabs and isolated from the stem wall footing to mitigate cracking. These recommendations are considered minimums unless superseded by the project structural engineer.

It is recommended that a vapor retarder/waterproofing be placed below the concrete slab on grade. Vapor/moisture transmission through slabs does occur and can impact various components of the structure. Waterproofing design and inspection of installation is not the responsibility of the geotechnical engineer. GeoConcepts, Inc. does not practice in the field of water and moisture vapor transmission evaluation/mitigation. Therefore, we recommend that a qualified person/firm be engaged/consulted to evaluate the general and specific water and moisture vapor transmission paths and any impact on the proposed development. This person/firm should provide recommendations for mitigation of potential adverse impact of water and moisture vapor transmission on various components of the structure as deemed necessary. The actual waterproofing design shall be provided by the architect, structural engineer or contractor with experience in waterproofing.

Decking

Exterior decking slabs on grade should be reinforced with minimum #4 reinforcing bars, placed at (16) inches on center each way and supported on compacted fill. Provisions for cracks should be incorporated into the design and construction of the decking. Concrete slabs should have sufficient control joints spaced at a maximum of approximately 8 feet. Decking planned adjacent to lawns, planters or adjacent to descending slopes should be provided with a 12-inch thickened edge. The deck reinforcement should be bent down into the edge. These recommendations are considered minimums unless superseded by the project structural engineer.

Slough Protection

Some surficial erosion/surficial slope failures may occur during inclement weather. In order to mitigate this possible occurrence from impacting the rear yard area and the proposed structure, it is recommended that the freeboard on the rear yard retaining wall be a minimum of (2) feet The sloughed materials behind these walls must be cleaned out each time deposition occurs, to allow them to function as envisioned.

Additionally all slopes should be planted and maintained as described in the Drainage an Maintenance section. Deep-rooted shrubs should be planted in staggered rows that do no exceed 10 feet on center over the slope face.

REVIEWS

Plan Review and Plan Notes

The final grading, building, and/or structural plans shall be reviewed and approved by t consultants to ensure that all recommendations are incorporated into the design or shown notes on the plan.

The final plans should reflect the following:

1. The Preliminary Geologic and Geotechnical Engineering Investigation by GeoConcepts, is a part of the plans.
2. Plans must be reviewed and signed by GeoConcepts, Inc.
3. The project geotechnical engineer and/or geologist must review all grading.
4. The project geotechnical engineer and/or geologist shall review all foundations.

Construction Review

Reviews will be required to verify all geologic and geotechnical work. It is required th footing excavations, seepage pits, and grading be reviewed by this office. This office sho notified at least **two working days** in advance of any field reviews so that staff personne be made available.

The property owner should take an active role in project safety by assigning responsibil authority to individuals qualified in appropriate construction safety principles and pra Generally, site safety should be assigned to the general contractor or construction manag is in control of the site and has the required expertise, which includes but not lim construction means, methods and safety precautions.

LIMITATIONS

General

This report is intended to be used only in its entirety. No portion or section of the report, by itself, is designed to completely represent any aspect of the project described herein. If any reader requires additional information or has questions regarding this report, GeoConcepts, Inc. should be contacted.

Subsurface conditions were interpreted on the basis of our field explorations and past experience. Although, between exploratory excavations, subsurface earth materials may vary in type, strength and many other properties from those interpreted. The findings, conclusions and recommendations presented herein are for the soil conditions encountered in the specific locations. Earth materials and conditions immediately adjacent to, or beneath those observed may have different characteristics, such as, earth type, physical properties and strength. Other soil conditions due to non-uniformity of the soil conditions or manmade alterations may be revealed during construction. If subsurface conditions differ from those encountered in the described exploration, this office should be advised immediately so that further recommendations may be made if required. If it is desired to minimize the possibility of such changes, additional explorations and testing can/should be performed.

Findings, conclusions and recommendations presented herein are based on experience and background. Therefore, findings, conclusions and recommendations are professional opinions and are not meant to indicate a control of nature.

This preliminary report provides information regarding the findings on the subject property. It is not designed to provide a guarantee that the site will be free of hazards in the future, such as but not limited to, landslides, slippage, liquefaction, expansive soils, differential settlement, debris flows, seepage, concentrated drainage or flooding. It may not be possible to eliminate all hazards, but homeowners must maintain their property and improve deficiencies to minimize these hazards.

This report may not be copied. If you wish to purchase additional copies, you may order them from this office.

CONSTRUCTION NOTICE

Construction can be challenging. GeoConcepts, Inc. has provided this report to advise you of the general site conditions, geotechnical feasibility of the proposed project, and overall site stability. It must be understood that the professional opinions provided herein are based upon subsurface data, laboratory testing, analyses, and interpretation thereof. Recommendations contained herein are based upon surface reconnaissance and minimum subsurface explorations deemed suitable by your consultants.

Although quantities for foundation concrete and steel may be estimated based on the findings provided in this report, provision should be made for possible changes in quantities during construction. If it is desired to minimize the possibility of such changes, additional exploration and testing should be considered. However, you must be aware that depths and magnitudes will most likely vary between explorations given in the report.

We appreciate the opportunity of serving you on this project. If you have any questions concerning this report, please contact the undersigned.

Respectfully submitted,
GEOCONCEPTS, INC.

REGISTERED PROFESSIONAL ENGINEER
SCOTT JOHN WALTER
No. 2476
GEOTECHNICAL
STATE OF CALIFORNIA

Scott J. Walter
Project Engineer
GE 2476
SJW/EL: 2793-4

PROFESSIONAL GEOLOGIST
EDMOND KEN LEE
No. 2545
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Edmond Lee
Project Geologist
CEG 2545

Distribution: (6) Addressee

APPENDIX I

SITE INFORMATION

Location Map
Regional Geologic Map
USGS Fault Map
Alquist-Priolo Fault Map
Seismic Hazard Map

Geologic Map
Cross Sections

Field Exploration
Exploration Logs 1 through 6 (In Pocket)
Exploration Logs A through D (In Pocket)

November 18, 2014
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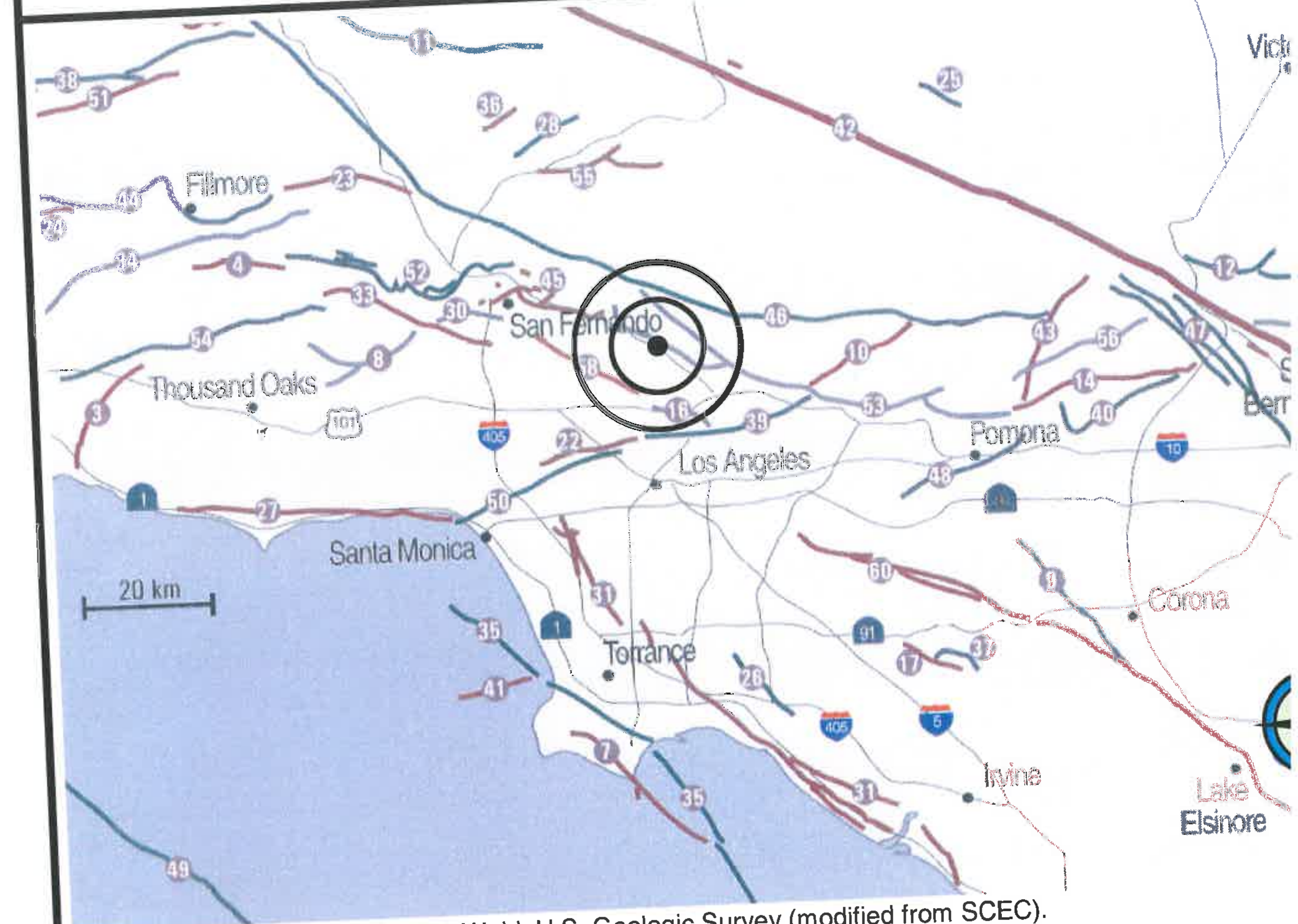
USGS FAULT MAP

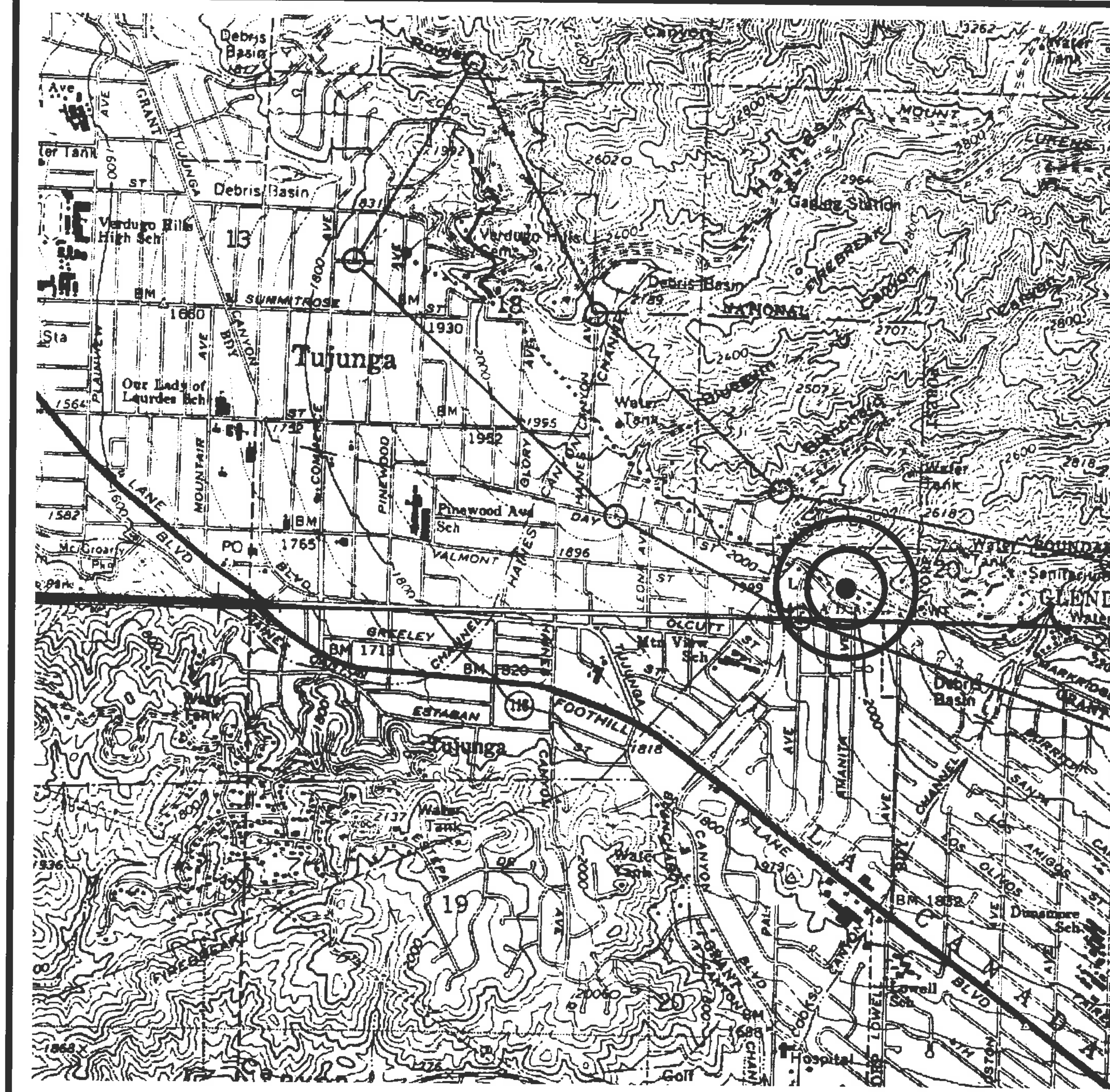


Lisa Wald, U.S. Geologic Survey (modified from SCEC).

1 Alamo thrust
2 Arrowhead fault
3 Bailey fault
4 Big Mountain fault
5 Big Pine fault
6 Blake Ranch fault
7 Cabrillo fault
8 Chatsworth fault
9 Chino fault
10 Clamshell-Sawpit fault
11 Clearwater fault
12 Cleghorn fault
13 Crafton Hills fault zone
14 Cucamonga fault zone
15 Dry Creek
16 Eagle Rock fault
17 El Modeno
18 Frazier Mountain thrust
19 Garlock fault zone
20 Grass Valley fault
21 Helendale fault
22 Hollywood fault
23 Holser fault
24 Lion Canyon fault
25 Llano fault
26 Los Alamitos fault
27 Malibu Coast fault
28 Mint Canyon fault
29 Mirage Valley fault zone
30 Mission Hills fault
31 Newport Inglewood fault zone
32 North Frontal fault zone
33 Northridge Hills fault
34 Oak Ridge fault
35 Palos Verdes fault zone
36 Pelona fault
37 Peralta Hills fault
38 Pine Mountain fault
39 Raymond fault
40 Red Hill (Etiwanda Ave) fault
41 Redondo Canyon fault
42 San Andreas Fault
43 San Antonio fault
44 San Cayetano fault
45 San Fernando fault zone
46 San Gabriel fault zone
47 San Jacinto fault
48 San Jose fault
49 Santa Cruz-Santa Catalina R
50 Santa Monica fault
51 Santa Ynez fault
52 Santa Susana fault zone
53 Sierra Madre fault zone
54 Simi fault
55 Soledad Canyon fault
56 Stoddard Canyon fault
57 Tunnel Ridge fault
58 Verdugo fault
59 Waterman Canyon fault
60 Whittier fault

Reference:	U. S. G. S: active fault (red) and potentially active fault (green)

ALQUIST-PRIOLO FAULT ZONE MAP



Reference:	CGS, AP Fault Zone Map of the Sunland and Burbank Quadrangles	

SEISMIC HAZARD MAP

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.

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.

Reference:	State of California, Seismic Hazard Map of the Sunland and Burbank Quadrangles

APPENDIX II

LABORATORY TESTING

Laboratory Procedures

Laboratory Recapitulation 1
Laboratory Recapitulation 2

Figures S.1 through S.4
Figures C.1 through C.5

LABORATORY PROCEDURES

Laboratory testing was performed on samples obtained as outlined in the Field Expl
section of this report. All samples were sent to the laboratory for examination, testing in (
conformance to specified test methods, and classification, using the Unified Soil Classi
System and group symbol.

Moisture and Density Tests

The dry unit weight and moisture content of the undisturbed samples were determine(
results are tabulated in the Laboratory Recapitulation - Table 1.

Shear Tests

Direct single-shear tests were performed with a direct shear machine. The desired norn
is applied to the specimen and allowed to come to equilibrium. The rate of deflection
sample is approximately 0.005 inches per minute. The samples are tested at higher
lower normal loads in order to determine the angle of internal friction and the cohesio
results are plotted on the Shear Test Diagrams and the results tabulated in the Lab
Recapitulation - Table 1.

Consolidation

Consolidation tests were performed on samples, within the brass ring, to predict th
behavior under a specific load. Porous stones are placed in contact with top and botton
samples to permit to allow the addition or release of water. Loads are applied in
increments and the results are recorded at selected time intervals. Samples are tested
and increased moisture content. The results are plotted on the Consolidation Test Cu
the load at which the water is added as noted on the drawing.

Expansion Index Tests

The sample is compacted into an expansion mold with a degree of saturation between 4
A vertical confining pressure of 144 psf is applied to the sample. The sample is inunda
distilled water. The deformation is recorded after 24 hours. The test results are show
Laboratory Recapitulation - Table 2.

LABORATORY RECAPITULATION
PROJECT: Day Street
PROJECT NO.: 2793

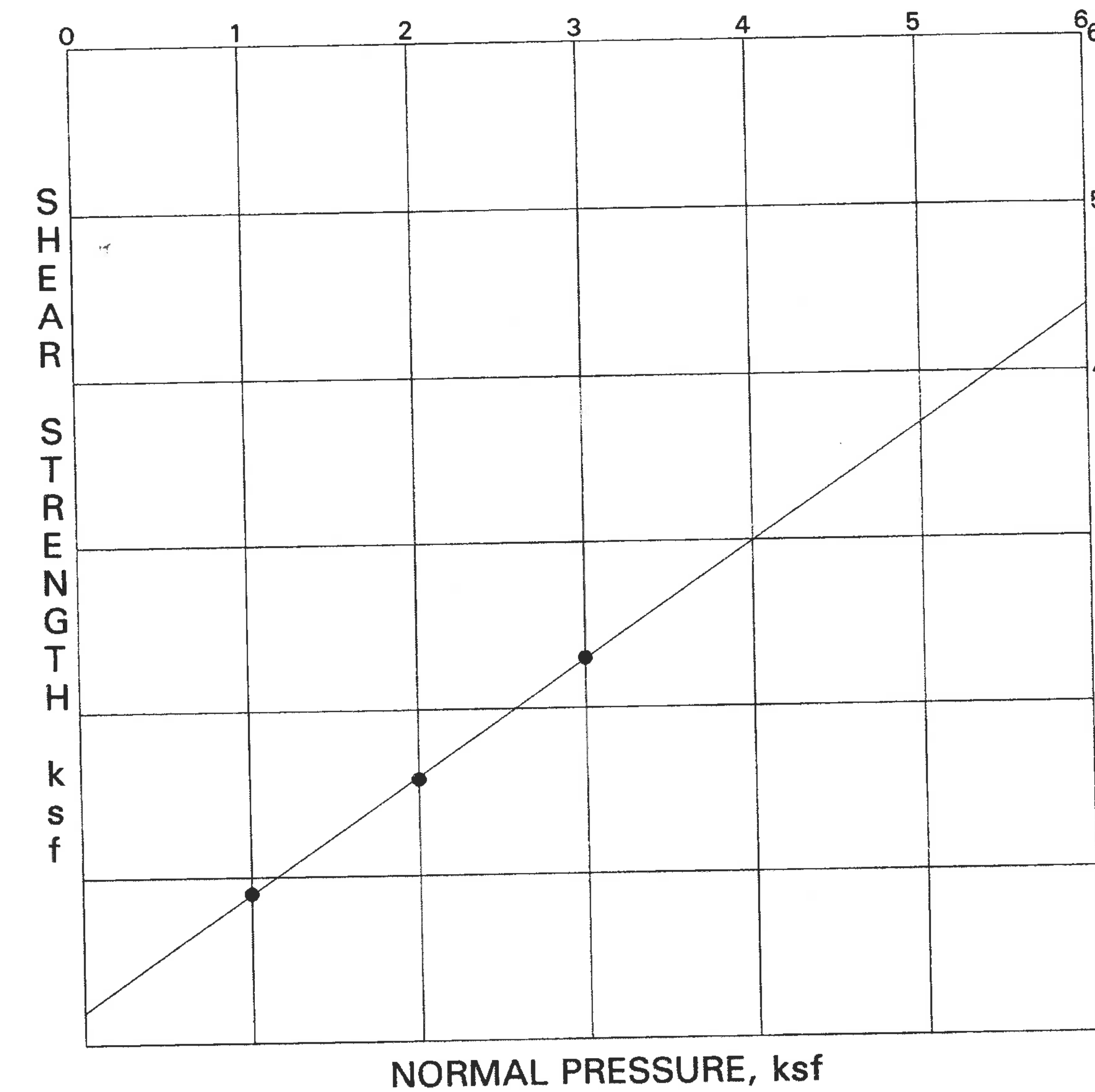
Explor -ation	Depth (ft)	Mat'l	Dry Dens. In Situ (P.C.F.)	Moisture Content (%)	Cohesion (K.S.F)	Friction Angle (degree)
TP 1	6.5	Qof_3	106.5	12.0		
TP 1	10.0	Qof_3	111.0	9.1		
TP 2	7.0	Qof_3	105.3	4.1	0.200	35.0
TP 4	8.0	Qof_1	107.7	12.4		
TP 6	6.0	Qof_3	112.4	1.8		
TP 6	11.0	Qof_3	109.8	5.7	0.200	33.0
TP 6	15.0	Qof_3	110.4	6.7		
TP 6	19.0	Qof_3	114.7	4.1		
TP-C	8.0	Qof_2	125.1	6.6	0.250	33.0
TP-D	5.0	wd	129.5	3.8	1.000	35.0

November 18, 2014
Project 2793

P

PROJECT: 2793
PROJECT LOCATION: Day Street

SAMPLE LOCATION: TP 2 @ 7.0
DESCRIPTION: Qof_3



Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 4.1 Saturated: 24.9	Dry Density: 105.3	Phi (deg): 35.0 Cohesion (psf): 200

SHEAR TEST DIAGRAM

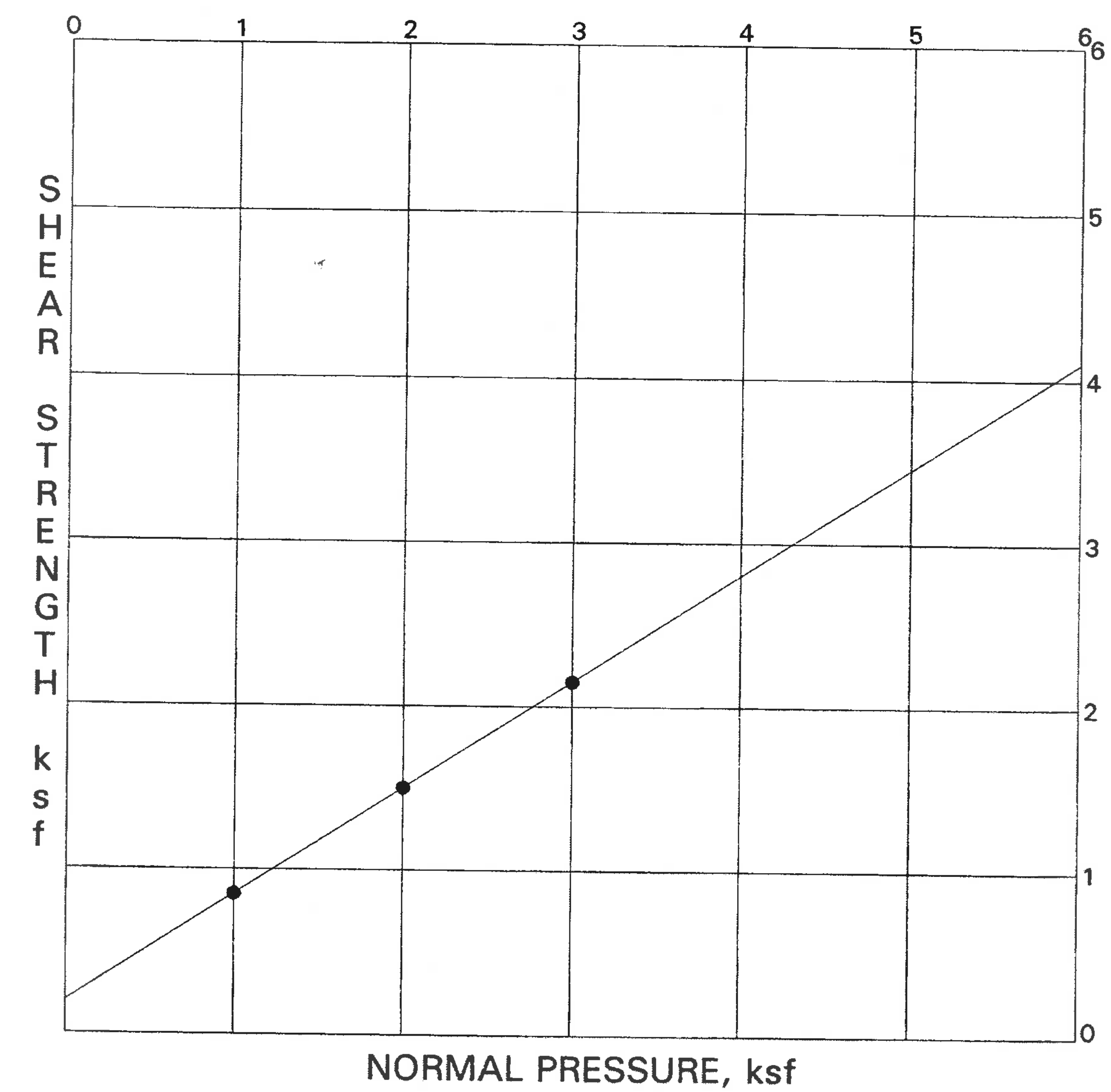
GeoConcepts, Inc.

14401 Gilmore St. #200

Figure S.

PROJECT: 2793
PROJECT LOCATION: Day Street

SAMPLE LOCATION: TP 6 @ 11.0
DESCRIPTION: Qof_3



Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 5.7 Saturated: 25.0	Dry Density: 109.8	Phi (deg): 33.0 Cohesion (psf): 200

SHEAR TEST DIAGRAM

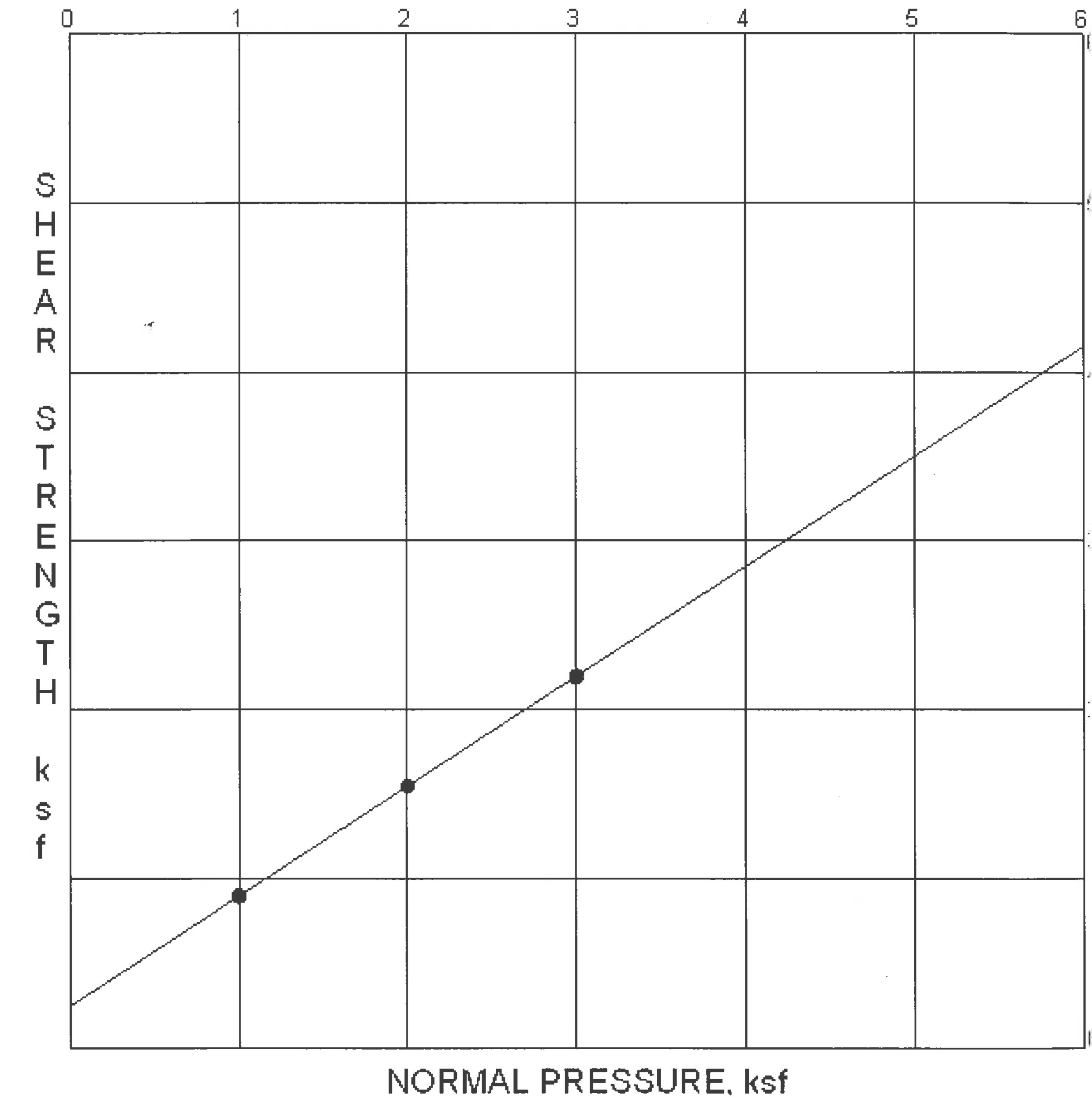
GeoConcepts, Inc.

14401 Gilmore St. #200

Figure S.2

PROJECT LOCATION: Day St PROJECT NO.: 2793

SAMPLE LOCATION: TP-C @ 8.0 DESCRIPTION: Qof



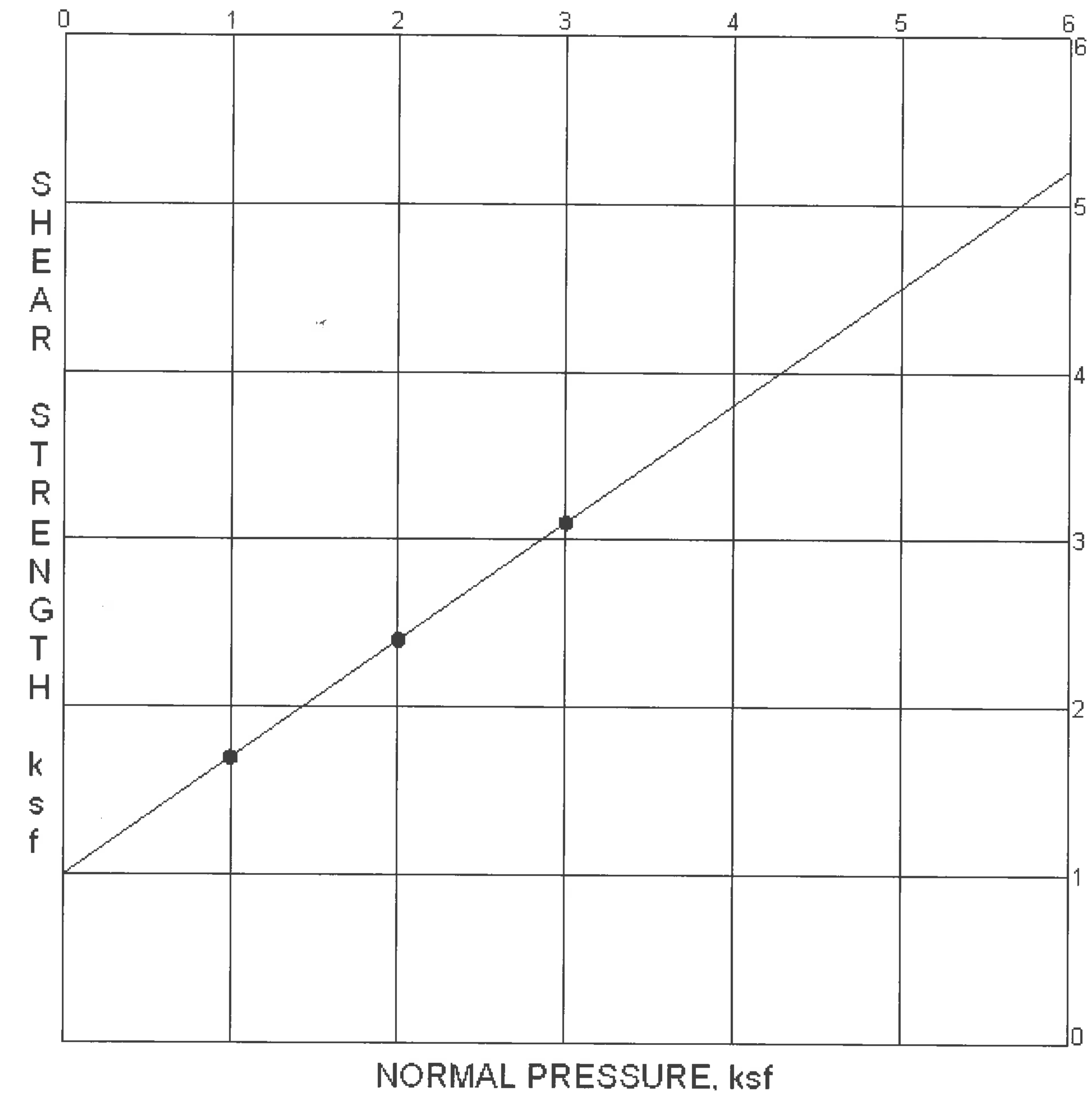
Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 6.6	Dry Density: 125.1	Phi (deg): 33.0
Saturated: 12.9		Cohesion (ksf): 0.250

SHEAR TEST DIAGRAM

Figur

PROJECT LOCATION:	Day St	PROJECT NO.:	2793
SAMPLE LOCATION:	TP-D @ 5.0	DESCRIPTION:	wd



Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 3.8	Dry Density: 129.5	Phi (deg): 35.0
Saturated: 11.1		Cohesion (ksf): 1.000

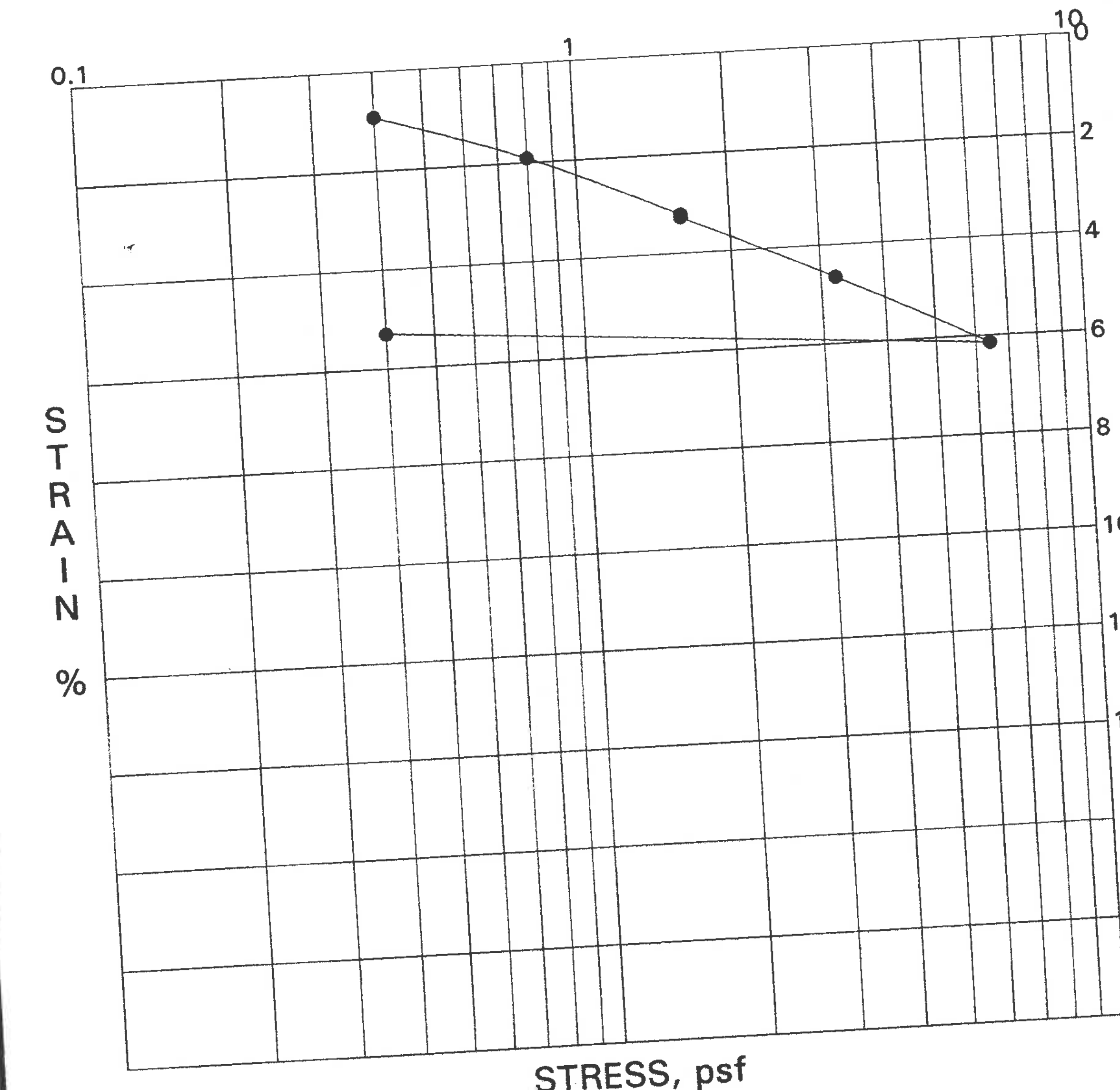
SHEAR TEST DIAGRAM

Figure S.4

November 18, 2014
Project 2793

PROJECT: 2793
PROJECT LOCATION: Day Street

SAMPLE LOCATION: TP 1 @ 6.5
DESCRIPTION: Qof3



Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 12.0	Dry Density: 106.5	1600 lbs.

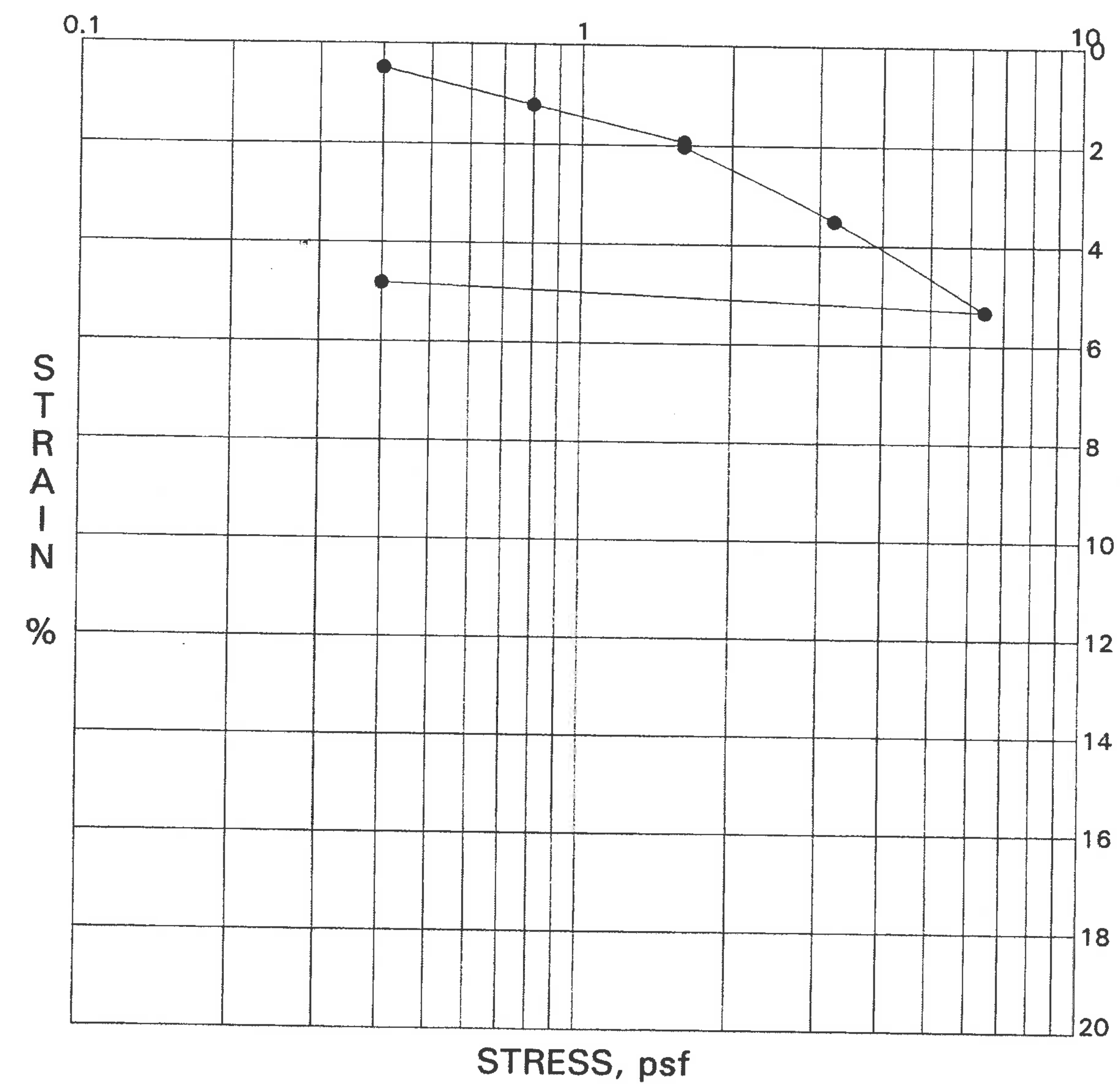
CONSOLIDATION TEST DIAGRAM

GeoConcepts, Inc.
14401 Gilmore St. #200

Figur

PROJECT: 2793
PROJECT LOCATION: Day Street

SAMPLE LOCATION: TP 2 @ 7.0
DESCRIPTION: **Qof$_3$**



Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 4.1	Dry Density: 105.3	1600 lbs.

CONSOLIDATION TEST DIAGRAM

GeoConcepts, Inc.

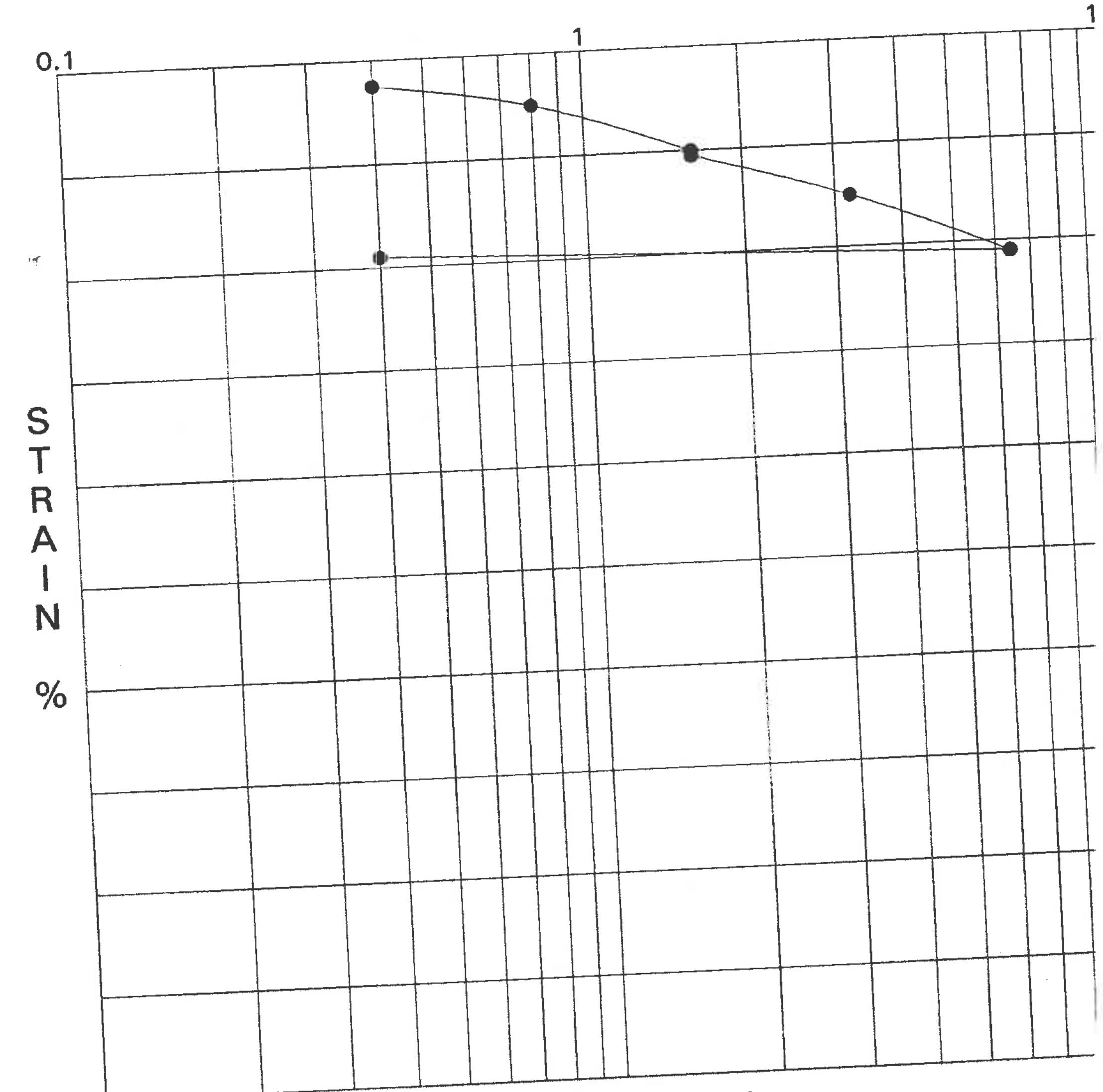
14401 Gilmore St. #200

Figure C.2

November 18, 2014
Project 2793

PROJECT: 2793
PROJECT LOCATION: Day Street

SAMPLE LOCATION: TP 6 @
DESCRIPTION: Qof_3



Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 5.7	Dry Density: 109.8	1600 lbs.

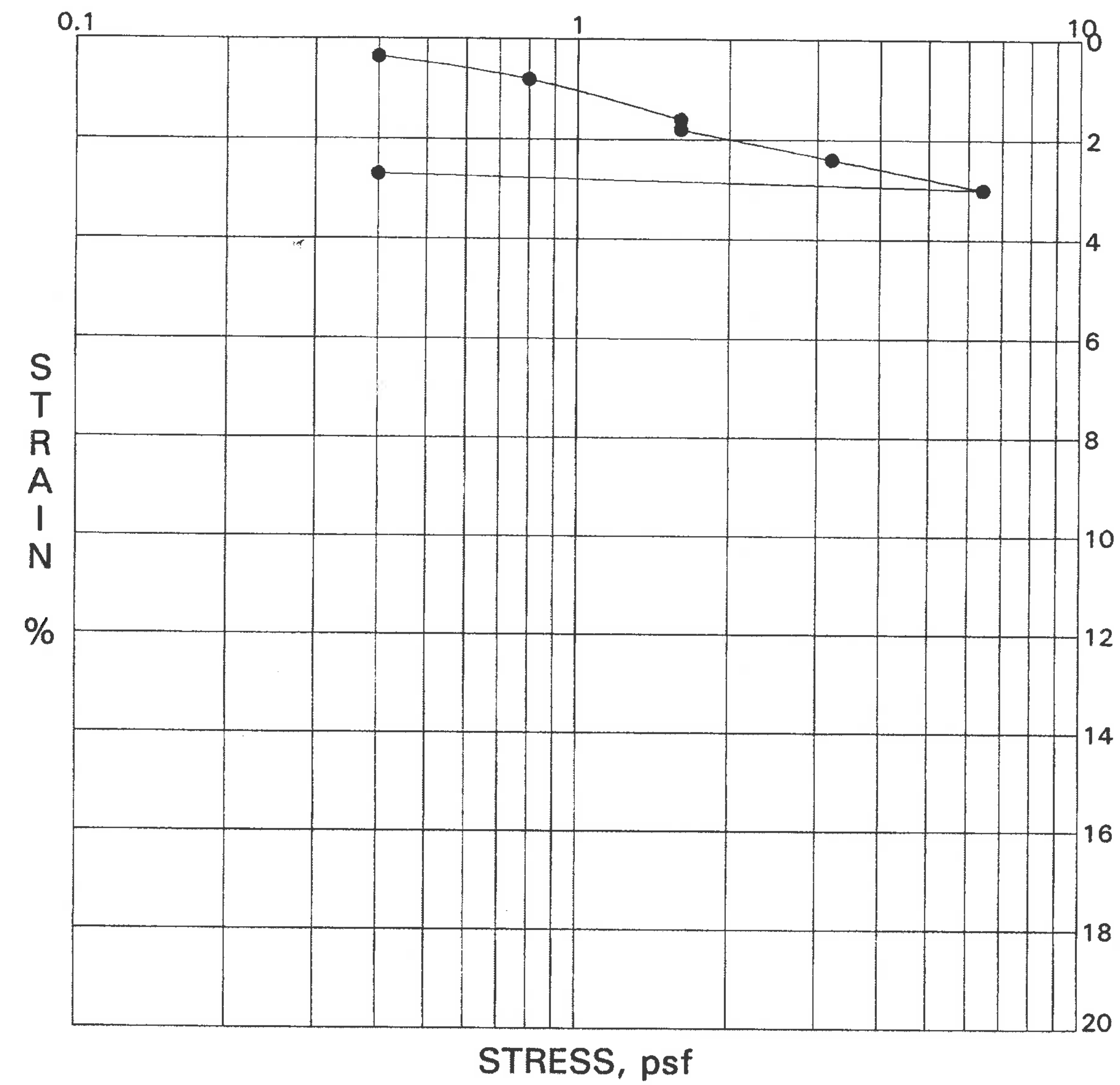
CONSOLIDATION TEST DIAGRAM

GeoConcepts, Inc.

14401 Gilmore St. #200

PROJECT: 2793
PROJECT LOCATION: Day Street

SAMPLE LOCATION: TP 6 @ 15.0
DESCRIPTION: **Qof3**



Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 6.7	Dry Density: 110.4	1600 lbs.

CONSOLIDATION TEST DIAGRAM

GeoConcepts, Inc.

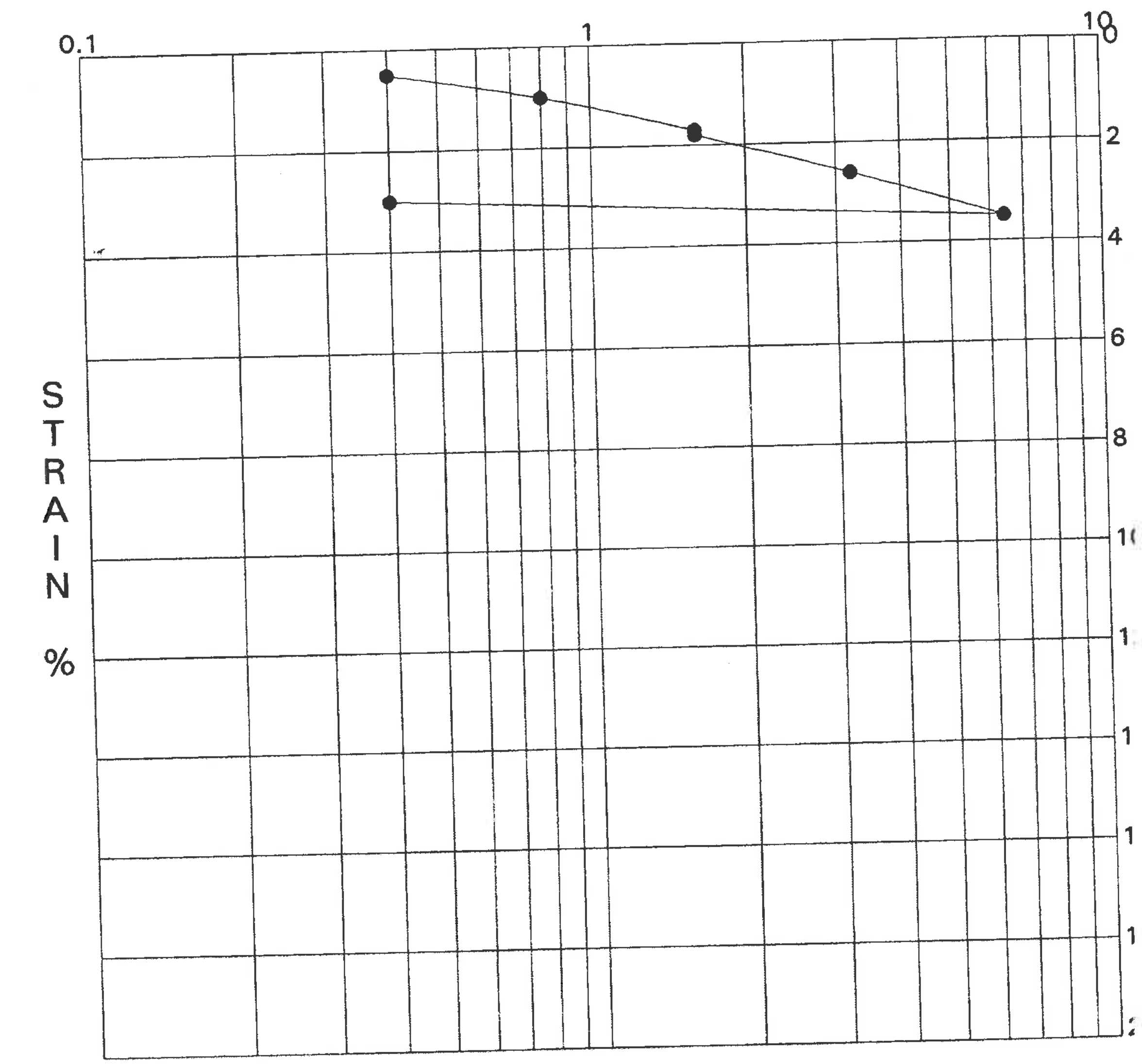
14401 Gilmore St. #200

Figure C.4

November 18, 2014
Project 2793

PROJECT: 2793
PROJECT LOCATION:Day Street

SAMPLE LOCATION:TP 6 @ 19
DESCRIPTION: Qof_3



Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 4.1	Dry Density: 114.7	1600 lbs.

CONSOLIDATION TEST DIAGRAM

GeoConcepts, Inc.

14401 Gilmore St. #200

Figure

APPENDIX III

ANALYSES

Bearing Capacity

Lateral Design

Slope Stability

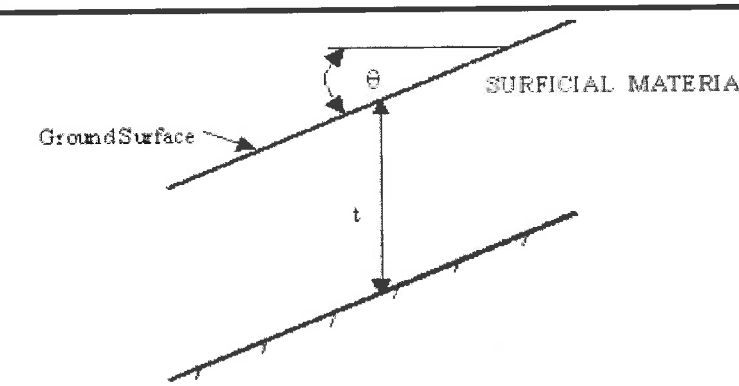
Seismic Evaluation

SURFICIAL STABILITY

CALCULATE THE SURFICIAL STABILITY OF THE EARTH MATERIAL USING THE INFINITE SLOPE ANALYSIS WITH PARALLEL SEEPAGE. THIS METHOD WAS RECOMMENDED BY THE ASCE AND THE BUILDING AND SAFETY ADVISORY COMMITTEE (8/16/78). MODIFIED FROM SKEMPTON & DeLORY, 1957.

CALCULATION PARAMETERS

EARTH MATERIAL:	Tm		
COHESION:	250 psf	SHEAR DIAGRAM:	TP-1@4.0
PHI ANGLE:	33 degrees	SLOPE ANGLE:	35 degrees
DENSITY:	130 pcf	SATURATION DEPTH (t):	4.0 feet



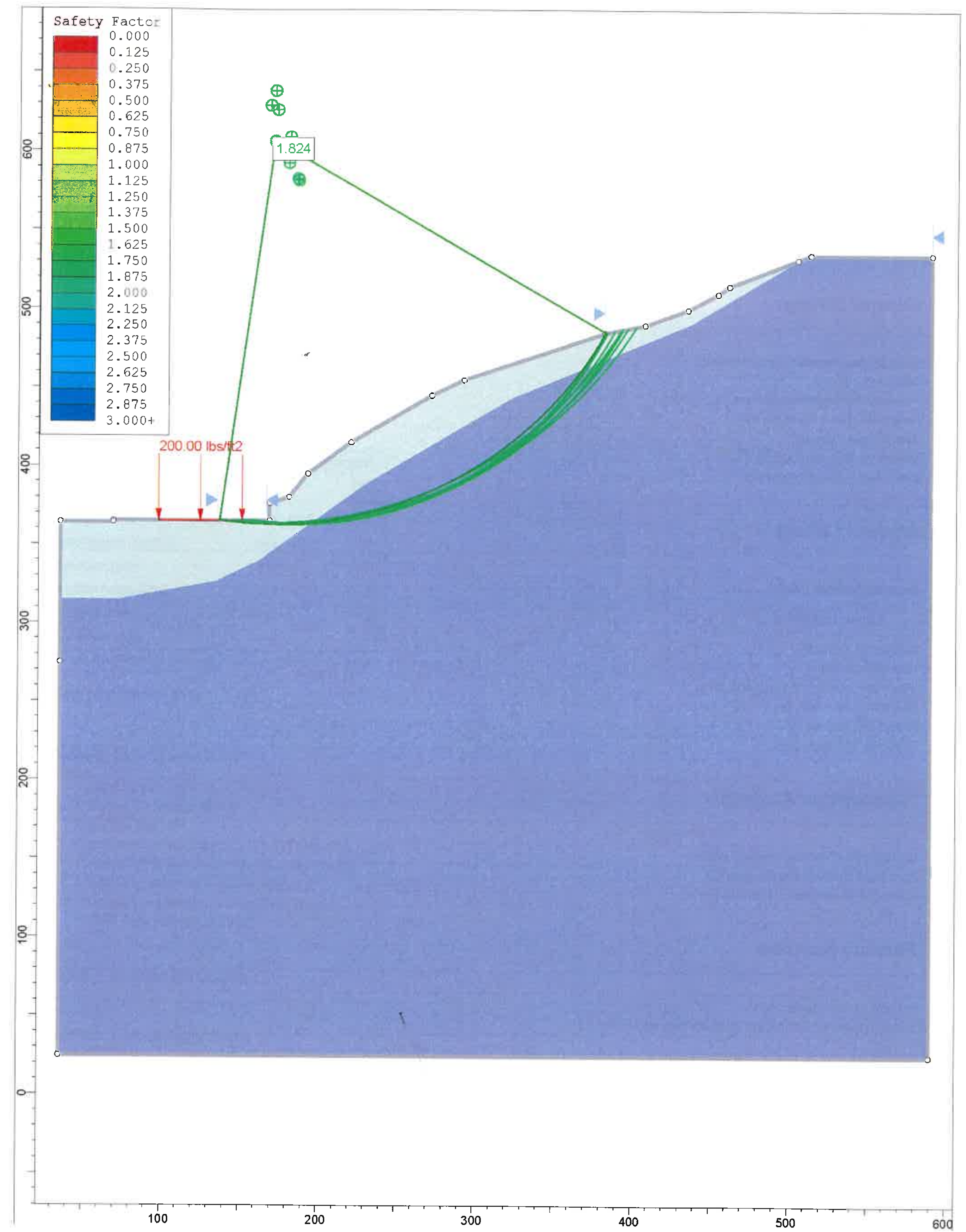
$$FS = \frac{C + (\gamma_{soil} - \gamma_{water}) \bullet t \bullet \cos^2\theta \tan\Phi}{\gamma_{soil} \bullet t \bullet \cos\Phi \sin\Phi}$$

SAFETY FACTOR = 1.51

CONCLUSIONS:

THE CALCULATION INDICATES THAT UNIFORM SLOPES IN COMPACTED FILL ARE SURFICIALLY STABLE.

Y-Y' Static Stability Analysis



November 18, 2014
Project 2793

Slide Analysis Information

2793 Y-Y' Static Stability Analysis

Project Summary

File Name: 2793 Y-Y'
Slide Modeler Version: 6.031
Project Title: 2793 Y-Y' Static Stability Analysis
Date Created: 11/17/2014, 10:17:13 AM

General Settings

Units of Measurement: Imperial Units
Time Units: days
Permeability Units: feet/second
Failure Direction: Right to Left
Data Output: Standard
Maximum Material Properties: 20
Maximum Support Properties: 20

Analysis Options

Analysis Methods Used

Bishop simplified

Number of slices: 25
Tolerance: 0.005
Maximum number of iterations: 50
Check malpha < 0.2: Yes
Initial trial value of FS: 1
Steffensen Iteration: Yes

Groundwater Analysis

Groundwater Method: Water Surfaces
Pore Fluid Unit Weight: 62.4 lbs/ft3
Advanced Groundwater Method: None

Random Numbers

Pseudo-random Seed: 10116
Random Number Generation Method: Park and Miller v.3

Surface Options

Surface Type: Circular
Search Method: Slope Search

Number of Surfaces: 5000
Upper Angle: Not Defined
Lower Angle: Not Defined
Composite Surfaces: Disabled
Reverse Curvature: Create Tension Crack
Minimum Elevation: Not Defined
Minimum Depth: Not Defined

Loading

1 Distributed Load present

Distributed Load 1

Distribution: Constant
Magnitude [psf]: 200
Orientation: Normal to boundary

Material Properties

Property	BEDROCK	FAN DEPOSITS
Color		
Strength Type	Mohr-Coulomb	Mohr-Coulomb
Unit Weight [lbs/ft3]	145	140
Cohesion [psf]	1000	250
Friction Angle [deg]	32	33
Water Surface	None	None
Ru Value	0	0

Global Minimums

Method: bishop simplified

FS: 1.824070
Center: 170.968, 606.170
Radius: 243.564
Left Slip Surface Endpoint: 136.753, 365.022
Right Slip Surface Endpoint: 382.379, 485.222
Resisting Moment=2.41009e+008 lb-ft
Driving Moment=1.32127e+008 lb-ft
Total Slice Area=9128.29 ft2

Valid / Invalid Surfaces

Method: bishop simplified

Number of Valid Surfaces: 5000
Number of Invalid Surfaces: 0

Slice Data

Global Minimum Query (bishop simplified) - Safety Factor: 1.82407

Slice Number	Width [ft]	Weight [lbs]	Base Material	Base Cohesion [psf]	Base Friction Angle [degrees]	Shear Stress [psf]	Shear Strength [psf]	Base Normal Stress [psf]	Pore Pressure [psf]	Effective Normal Stress [psf]
1	9.69058	798.407	FAN DEPOSITS	250	33	248.316	452.946	312.51	0	312.51
2	9.69058	2129.54	FAN DEPOSITS	250	33	256.154	467.243	334.525	0	334.525
3	9.69058	2931.89	FAN DEPOSITS	250	33	248.395	453.09	312.731	0	312.731
4	9.69058	15460.3	FAN DEPOSITS	250	33	705.357	1286.62	1596.26	0	1596.26
5	9.69058	24520.9	FAN DEPOSITS	250	33	1023.88	1867.63	2490.93	0	2490.93
6	9.69058	38207.2	FAN DEPOSITS	250	33	1498.86	2734.03	3825.08	0	3825.08
7	9.60119	48194.9	BEDROCK	1000	32	2179.25	3975.11	4761.18	0	4761.18
8	9.60119	56090.9	BEDROCK	1000	32	2417.73	4410.11	5457.32	0	5457.32
9	9.60119	63306.2	BEDROCK	1000	32	2626.62	4791.14	6067.1	0	6067.1
10	9.60119	68552.5	BEDROCK	1000	32	2764.29	5042.25	6468.96	0	6468.96
11	9.60119	72804	BEDROCK	1000	32	2864.84	5225.66	6762.48	0	6762.48
12	9.60119	76448.9	BEDROCK	1000	32	2941.82	5366.09	6987.2	0	6987.2
13	9.60119	79476.3	BEDROCK	1000	32	2995.45	5463.91	7143.75	0	7143.75
14	9.60119	81858	BEDROCK	1000	32	3025.36	5518.47	7231.06	0	7231.06
15	9.60119	82968.5	BEDROCK	1000	32	3012.93	5495.79	7194.77	0	7194.77
16	9.60119	82755.4	BEDROCK	1000	32	2958.01	5395.62	7034.47	0	7034.47
17	9.60119	81039.8	BEDROCK	1000	32	2856.86	5211.12	6739.19	0	6739.19
18	9.60119	77257.2	BEDROCK	1000	32	2695	4915.86	6266.68	0	6266.68
19	9.60119	72542.6	BEDROCK	1000	32	2507.85	4574.49	5720.38	0	5720.38
20	9.60119	66801.3	BEDROCK	1000	32	2294.13	4184.65	5096.51	0	5096.51
21	9.60119	59866.9	BEDROCK	1000	32	2051.03	3741.23	4386.88	0	4386.88
22	9.60119	51629	BEDROCK	1000	32	1778.03	3243.25	3589.96	0	3589.96
23	9.60119	41869	BEDROCK	1000	32	1472.7	2686.31	2698.66	0	2698.66
24	9.60119	30270.1	BEDROCK	1000	32	1131.93	2064.72	1703.91	0	1703.91
25	14.661	18419.7	FAN DEPOSITS	250	33	377.372	688.353	675.006	0	675.006

Interslice Data

Global Minimum Query (bishop simplified) - Safety Factor: 1.82407

Slice Number	X coordinate [ft]	Y coordinate - Bottom [ft]	Interslice Normal Force [lbs]	Interslice Shear Force [lbs]	Interslice Force Angle [degrees]
1	136.753	365.022	0	0	0
2	146.444	363.844	2770.34	0	0
3	156.134	363.059	5511.45	0	0
4	165.825	362.661	8039.09	0	0
5	175.515	362.649	14882.4	0	0
6	185.206	363.023	23856.7	0	0
7	194.897	363.785	35444.5	0	0
8	204.498	364.926	50903.4	0	0
9	214.099	366.456	65727.7	0	0
10	223.7	368.383	79211.4	0	0
11	233.301	370.718	90607.6	0	0
12	242.903	373.472	99447.7	0	0
13	252.504	376.66	105373	0	0

14	262.105	380.3	108079	0	0
15	271.706	384.416	107319	0	0
16	281.307	389.033	102979	0	0
17	290.908	394.186	95089.8	0	0
18	300.51	399.913	83878.8	0	0
19	310.111	406.264	69910.5	0	0
20	319.712	413.301	53694.4	0	0
21	329.313	421.103	35925.7	0	0
22	338.914	429.77	17566	0	0
23	348.516	439.437	-95.0947	0	0
24	358.117	450.291	-15268.3	0	0
25	367.718	462.6	-25391.1	0	0
26	382.379	485.222	0	0	0

List Of Coordinates

Distributed Load

X	Y
151	365.021
98	365.023

External Boundary

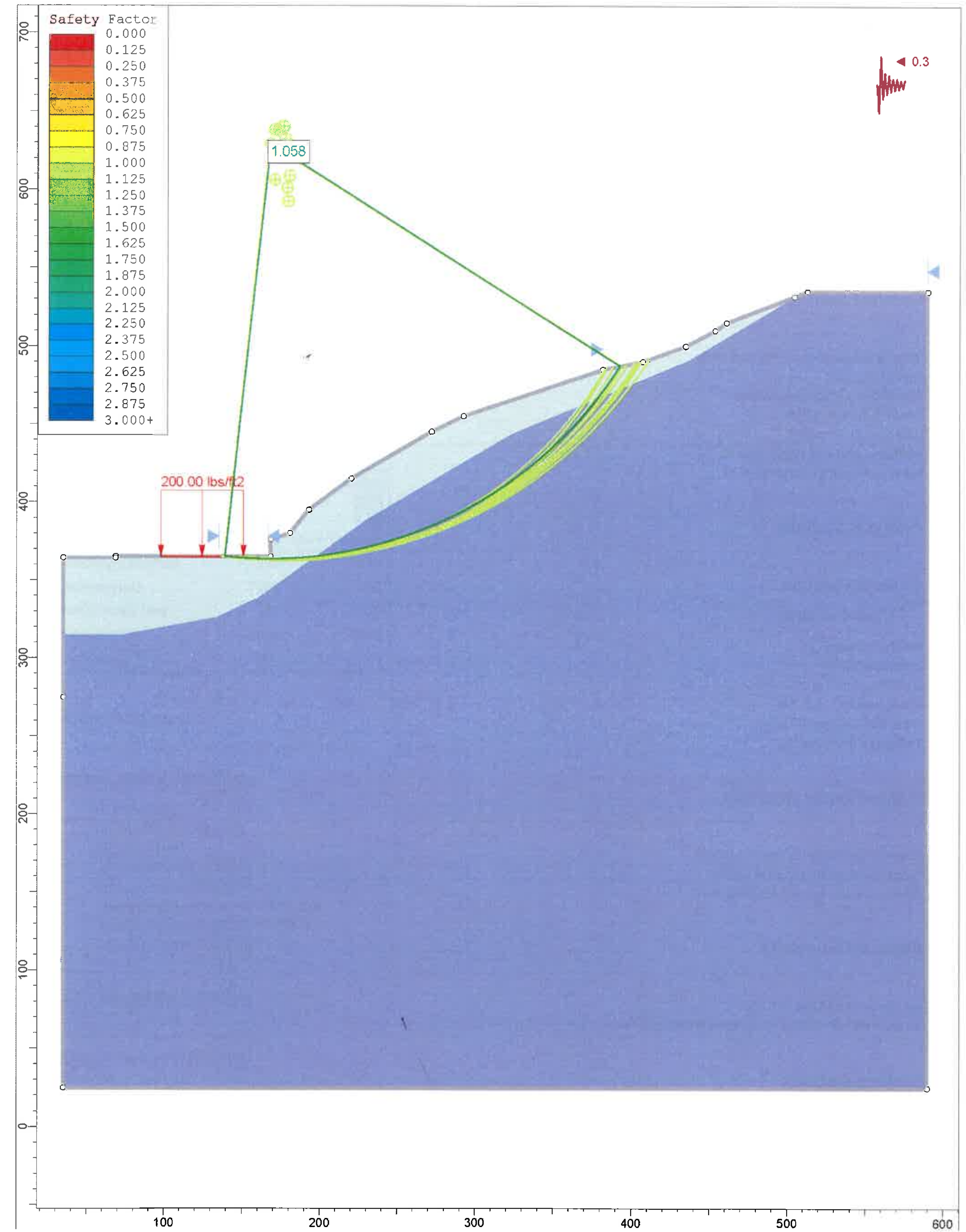
X	Y
35.1599	24.6325
589.717	24.6325
589.717	535.089
512.641	535.089
504.387	531.903
460.718	515.05
453.571	510.003
434.587	499.998
407.216	490.05
381.475	485.046
292.269	455.099
271.749	445.058
220.124	415.024
192.737	395.025
180.683	380.056
168.316	375.961
168.287	365.021
68.8184	365.024
68.8184	364.071
35.1599	364.071
35.1599	274.673

Material Boundary

X	Y

35.1599	364.071
35.16	315
75	315
134	326
159	338
230	389
323	444
436	491
504.387	531.903

Y-Y' Pseudo-Static Stability Analysis



November 18, 2014
Project 2793

Slide Analysis Information

2793 Y-Y' Pseudo-Static Stability Analysis

Project Summary

File Name: 2793 Y-Y'
Slide Modeler Version: 6.031
Project Title: 2793 Y-Y' Pseudo-Static Stability Analysis
Date Created: 11/17/2014, 10:17:13 AM

General Settings

Units of Measurement: Imperial Units
Time Units: days
Permeability Units: feet/second
Failure Direction: Right to Left
Data Output: Standard
Maximum Material Properties: 20
Maximum Support Properties: 20

Analysis Options

Analysis Methods Used

Bishop simplified

Number of slices: 25
Tolerance: 0.005
Maximum number of iterations: 50
Check malpha < 0.2: Yes
Initial trial value of FS: 1
Steffensen Iteration: Yes

Groundwater Analysis

Groundwater Method: Water Surfaces
Pore Fluid Unit Weight: 62.4 lbs/ft3
Advanced Groundwater Method: None

Random Numbers

Pseudo-random Seed: 10116
Random Number Generation Method: Park and Miller v.3

Surface Options

Surface Type: Circular
Search Method: Slope Search

Number of Surfaces: 5000
Upper Angle: Not Defined
Lower Angle: Not Defined
Composite Surfaces: Disabled
Reverse Curvature: Create Tension Crack
Minimum Elevation: Not Defined
Minimum Depth: Not Defined

Loading

Seismic Load Coefficient (Horizontal): 0.3
1 Distributed Load present

Distributed Load 1

Distribution: Constant
Magnitude [psf]: 200
Orientation: Normal to boundary

Material Properties

Property	BEDROCK	FAN DEPOSITS
Color		
Strength Type	Mohr-Coulomb	Mohr-Coulomb
Unit Weight [lbs/ft3]	145	140
Cohesion [psf]	1000	250
Friction Angle [deg]	32	33
Water Surface	None	None
Ru Value	0	0

Global Minimums

Method: bishop simplified

FS: 1.057700
Center: 168.061, 628.859
Radius: 265.432
Left Slip Surface Endpoint: 139.012, 365.022
Right Slip Surface Endpoint: 392.528, 487.195
Resisting Moment=2.47963e+008 lb-ft
Driving Moment=2.34437e+008 lb-ft
Total Slice Area=9532.74 ft2

Valid / Invalid Surfaces

Method: bishop simplified

Number of Valid Surfaces: 5000
Number of Invalid Surfaces: 0

Slice Data

Global Minimum Query (bishop simplified) - Safety Factor: 1.0577

Slice Number	Width [ft]	Weight [lbs]	Base Material	Base Cohesion [psf]	Base Friction Angle [degrees]	Shear Stress [psf]	Shear Strength [psf]	Base Normal Stress [psf]	Pore Pressure [psf]	Effective Normal Stress [psf]
1	9.58631	590.187	FAN DEPOSITS	250	33	420.66	444.932	300.169	0	300.169
2	9.58631	1536.27	FAN DEPOSITS	250	33	378.374	400.206	231.297	0	231.297
3	9.58631	2015.17	FAN DEPOSITS	250	33	369.778	391.114	217.296	0	217.296
4	9.58631	17795.2	FAN DEPOSITS	250	33	1361.91	1440.49	1833.2	0	1833.2
5	9.58631	24907.4	FAN DEPOSITS	250	33	1773.73	1876.07	2503.93	0	2503.93
6	9.58631	38745.9	FAN DEPOSITS	250	33	2576.28	2724.93	3811.05	0	3811.05
7	10.1091	50936.2	BEDROCK	1000	32	3647.78	3858.26	4574.18	0	4574.18
8	10.1091	59572.4	BEDROCK	1000	32	4030.07	4262.61	5221.28	0	5221.28
9	10.1091	67148.8	BEDROCK	1000	32	4339.56	4589.95	5745.12	0	5745.12
10	10.1091	72573.6	BEDROCK	1000	32	4524.21	4785.26	6057.69	0	6057.69
11	10.1091	77215.8	BEDROCK	1000	32	4659.28	4928.12	6286.31	0	6286.31
12	10.1091	81216.9	BEDROCK	1000	32	4753.89	5028.19	6446.44	0	6446.44
13	10.1091	84555.6	BEDROCK	1000	32	4808.66	5086.12	6539.16	0	6539.16
14	10.1091	86995.6	BEDROCK	1000	32	4814.17	5091.95	6548.48	0	6548.48
15	10.1091	87721	BEDROCK	1000	32	4735.48	5008.72	6415.3	0	6415.3
16	10.1091	87240	BEDROCK	1000	32	4600.11	4865.54	6186.16	0	6186.16
17	10.1091	84291	BEDROCK	1000	32	4356.59	4607.97	5773.95	0	5773.95
18	10.1091	80214.2	BEDROCK	1000	32	4069.18	4303.97	5287.46	0	5287.46
19	10.1091	75164.7	BEDROCK	1000	32	3747.59	3963.83	4743.11	0	4743.11
20	10.1091	68989.5	BEDROCK	1000	32	3388.77	3584.3	4135.74	0	4135.74
21	10.1091	61622	BEDROCK	1000	32	2994.09	3166.85	3467.68	0	3467.68
22	10.1091	52915.4	BEDROCK	1000	32	2562.88	2710.76	2737.79	0	2737.79
23	10.1091	42658.6	BEDROCK	1000	32	2093.88	2214.7	1943.92	0	1943.92
24	10.1091	30560.5	BEDROCK	1000	32	1585.89	1677.39	1084.06	0	1084.06
25	14.0335	16990.3	FAN DEPOSITS	250	33	521.153	551.224	463.845	0	463.845

Interslice Data

Global Minimum Query (bishop simplified) - Safety Factor: 1.0577

Slice Number	X coordinate [ft]	Y coordinate - Bottom [ft]	Interslice Normal Force [lbs]	Interslice Shear Force [lbs]	Interslice Force Angle [degrees]
1	139.012	365.022	0	0	0
2	148.599	364.142	4119.12	0	0
3	158.185	363.611	7407.79	0	0
4	167.771	363.427	10387.6	0	0
5	177.358	363.59	17805.1	0	0
6	186.944	364.1	26058.3	0	0
7	196.53	364.958	35856.5	0	0
8	206.639	366.246	51559.1	0	0
9	216.749	367.931	65625.5	0	0
10	226.858	370.021	77335	0	0
11	236.967	372.527	86113.8	0	0
12	247.076	375.461	91603.6	0	0

13	257.185	378.837	93525.3	0	0
14	267.294	382.674	91671.5	0	0
15	277.403	386.995	85943.2	0	0
16	287.512	391.824	76509.9	0	0
17	297.621	397.195	63611.3	0	0
18	307.73	403.146	48001.9	0	0
19	317.84	409.723	30290	0	0
20	327.949	416.986	11171.6	0	0
21	338.058	425.008	-8447.47	0	0
22	348.167	433.882	-27442.2	0	0
23	358.276	443.731	-44375.4	0	0
24	368.385	454.72	-57369.9	0	0
25	378.494	467.082	-63908.3	0	0
26	392.528	487.195	0	0	0

List Of Coordinates

Distributed Load

X	Y
151	365.021
98	365.023

External Boundary

X	Y
35.1599	24.6325
589.717	24.6325
589.717	535.089
512.641	535.089
504.387	531.903
460.718	515.05
453.571	510.003
434.587	499.998
407.216	490.05
381.475	485.046
292.269	455.099
271.749	445.058
220.124	415.024
192.737	395.025
180.683	380.056
168.316	375.961
168.287	365.021
68.8184	365.024
68.8184	364.071
35.1599	364.071
35.1599	274.673

Material Boundary

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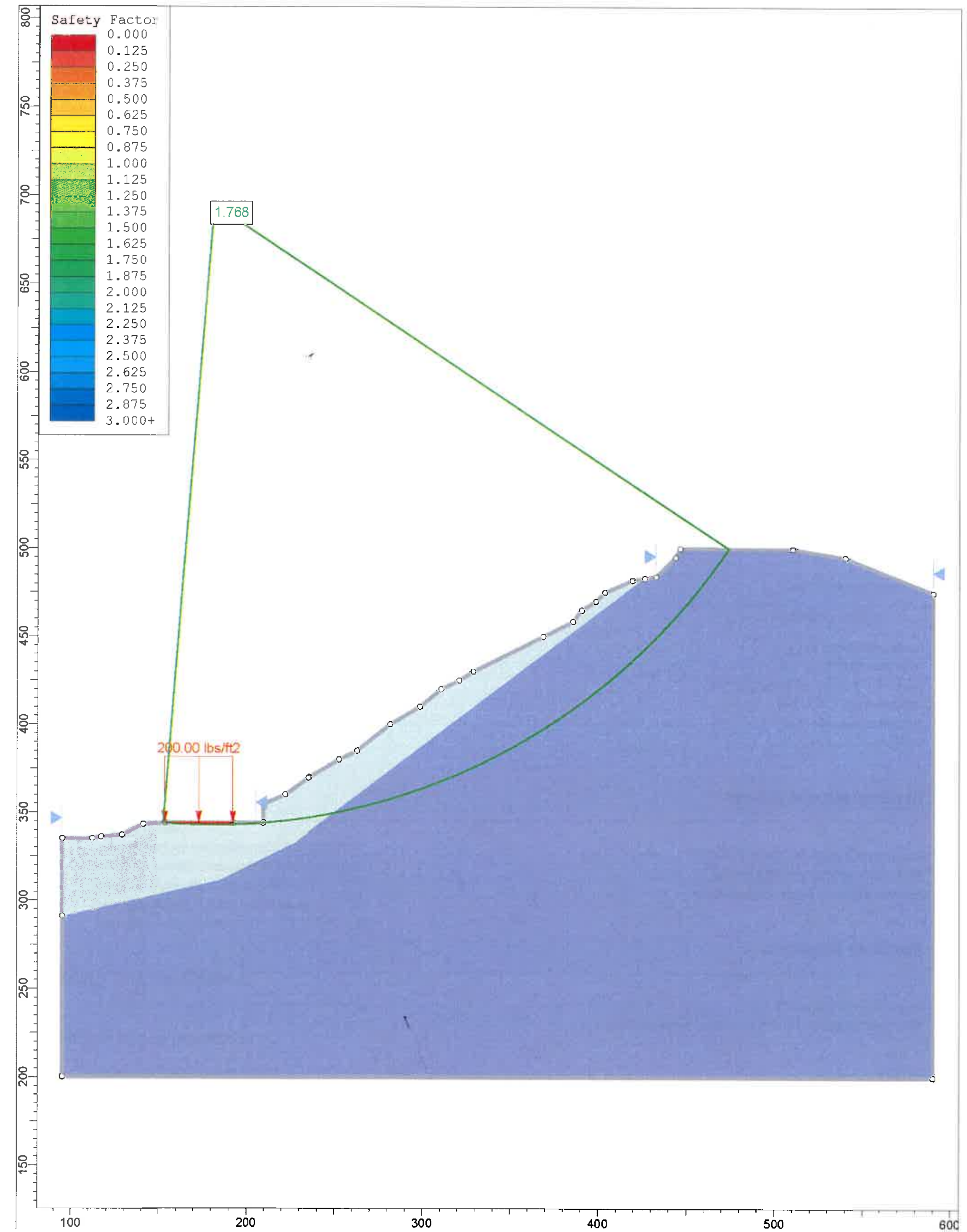
X	Y
35.1599	364.071
35.16	315
75	315
134	326
159	338
230	389
323	444
436	491
504.387	531.903

Z-Z' Slope Stability Analysis



Slide Analysis Information

2793 Z-Z' Slope Stability Analysis

Project Summary

File Name: 2793 Z-Z'
Slide Modeler Version: 6.031
Project Title: 2793 Z-Z' Slope Stability Analysis
Date Created: 11/17/2014, 10:17:37 AM

General Settings

Units of Measurement: Imperial Units
Time Units: days
Permeability Units: feet/second
Failure Direction: Right to Left
Data Output: Standard
Maximum Material Properties: 20
Maximum Support Properties: 20

Analysis Options

Analysis Methods Used

Bishop simplified

Number of slices: 25
Tolerance: 0.005
Maximum number of iterations: 50
Check malpha < 0.2: Yes
Initial trial value of FS: 1
Steffensen Iteration: Yes

Groundwater Analysis

Groundwater Method: Water Surfaces
Pore Fluid Unit Weight: 62.4 lbs/ft3
Advanced Groundwater Method: None

Random Numbers

Pseudo-random Seed: 10116
Random Number Generation Method: Park and Miller v.3

Surface Options

Surface Type: Circular
Search Method: Slope Search

Number of Surfaces: 5000
Upper Angle: Not Defined
Lower Angle: Not Defined
Composite Surfaces: Disabled
Reverse Curvature: Create Tension Crack
Minimum Elevation: Not Defined
Minimum Depth: Not Defined

Loading

1 Distributed Load present

Distributed Load 1

Distribution: Constant
Magnitude [psf]: 200
Orientation: Normal to boundary

Material Properties

Property	BEDROCK	FAN DEPOSITS
Color		
Strength Type	Mohr-Coulomb	Mohr-Coulomb
Unit Weight [lbs/ft3]	145	135
Cohesion [psf]	1000	250
Friction Angle [deg]	35	33
Water Surface	None	None
Ru Value	0	0

Global Minimums

Method: bishop simplified

FS: 1.768490
Center: 180.804, 694.667
Radius: 351.807
Left Slip Surface Endpoint: 152.315, 344.015
Right Slip Surface Endpoint: 473.902, 500.086
Resisting Moment=4.3864e+008 lb-ft
Driving Moment=2.48031e+008 lb-ft
Total Slice Area=10565.3 ft2

Valid / Invalid Surfaces

Method: bishop simplified

Number of Valid Surfaces: 4909
Number of Invalid Surfaces: 91

Error Codes:

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Project 2793

Error Code -113 reported for 91 surfaces

Error Codes

The following errors were encountered during the computation:

-113 = Surface intersects outside slope limits.

Slice Data

Global Minimum Query (bishop simplified) - Safety Factor: 1.76849

Slice Number	Width [ft]	Weight [lbs]	Base Material	Base Cohesion [psf]	Base Friction Angle [degrees]	Shear Stress [psf]	Shear Strength [psf]	Base Normal Stress [psf]	Pore Pressure [psf]	Effective Normal Str [psf]
1	12.102	735.12	FAN DEPOSITS	250	33	236.831	418.834	259.982	0	259.
2	12.102	1662.22	FAN DEPOSITS	250	33	268.129	474.183	345.212	0	345.
3	12.102	1903.54	FAN DEPOSITS	250	33	272.064	481.143	355.929	0	355.
4	12.102	1464.19	FAN DEPOSITS	250	33	203.638	360.132	169.589	0	169.
5	12.102	5769.61	FAN DEPOSITS	250	33	308.079	544.834	454.004	0	454.
6	12.102	22901.4	FAN DEPOSITS	250	33	804.195	1422.21	1805.05	0	180
7	12.102	33167.3	FAN DEPOSITS	250	33	1090.22	1928.04	2583.96	0	258
8	12.102	42746.3	FAN DEPOSITS	250	33	1349.51	2386.6	3290.07	0	329
9	13.2218	55321.6	BEDROCK	1000	35	2045.93	3618.21	3739.2	0	37
10	13.2218	65731	BEDROCK	1000	35	2298.8	4065.4	4377.85	0	437
11	13.2218	77989.3	BEDROCK	1000	35	2592.73	4585.22	5120.22	0	512
12	13.2218	85695.4	BEDROCK	1000	35	2756.54	4874.91	5533.95	0	553
13	13.2218	95172.1	BEDROCK	1000	35	2959.58	5233.98	6046.75	0	604
14	13.2218	99740.1	BEDROCK	1000	35	3028.72	5356.26	6221.38	0	622
15	13.2218	102443	BEDROCK	1000	35	3046.15	5387.08	6265.4	0	62
16	13.2218	102553	BEDROCK	1000	35	2995.86	5298.15	6138.39	0	613
17	13.2218	101258	BEDROCK	1000	35	2909.63	5145.65	5920.6	0	59
18	13.2218	98449.8	BEDROCK	1000	35	2786.39	4927.7	5609.33	0	560
19	13.2218	97781.3	BEDROCK	1000	35	2712.85	4797.64	5423.59	0	542
20	13.2218	98779.8	BEDROCK	1000	35	2674.41	4729.67	5326.52	0	532
21	13.2218	91534.7	BEDROCK	1000	35	2447.73	4328.79	4754.01	0	475
22	13.2218	75262.9	BEDROCK	1000	35	2029.65	3589.41	3698.05	0	369
23	13.2218	66732.1	BEDROCK	1000	35	1788.2	3162.42	3088.26	0	308
24	13.2218	51359.7	BEDROCK	1000	35	1414.05	2500.73	2143.27	0	214
25	13.2218	17840.4	BEDROCK	1000	35	707.254	1250.77	358.14	0	35

Interslice Data

Global Minimum Query (bishop simplified) - Safety Factor: 1.76849

Slice Number	X coordinate [ft]	Y coordinate - Bottom [ft]	Interslice Normal Force [lbs]	Interslice Shear Force [lbs]	Interslice Force Angle [degrees]
1	152.315	344.015	0	0	0
2	164.417	343.242	3059.53	0	0
3	176.519	342.886	6418.5	0	0
4	188.621	342.947	9680.54	0	0

5	200.723	343.424	12057.3	0	0
6	212.825	344.32	15368.9	0	0
7	224.927	345.638	22696.6	0	0
8	237.029	347.382	31348.2	0	0
9	249.131	349.559	40473.5	0	0
10	262.353	352.442	56671.1	0	0
11	275.575	355.865	71997.2	0	0
12	288.796	359.845	85807.8	0	0
13	302.018	364.401	96939.9	0	0
14	315.24	369.559	104778	0	0
15	328.462	375.347	108708	0	0
16	341.684	381.8	108445	0	0
17	354.906	388.96	103997	0	0
18	368.127	396.878	95482.1	0	0
19	381.349	405.617	83204.3	0	0
20	394.571	415.254	66712.2	0	0
21	407.793	425.884	45353.8	0	0
22	421.015	437.632	21780.5	0	0
23	434.236	450.658	370.794	0	0
24	447.458	465.181	-20900.1	0	0
25	460.68	481.506	-37242.7	0	0
26	473.902	500.086	0	0	0

List Of Coordinates

Distributed Load

X	Y
192.047	344.082
153.275	344.081

External Boundary

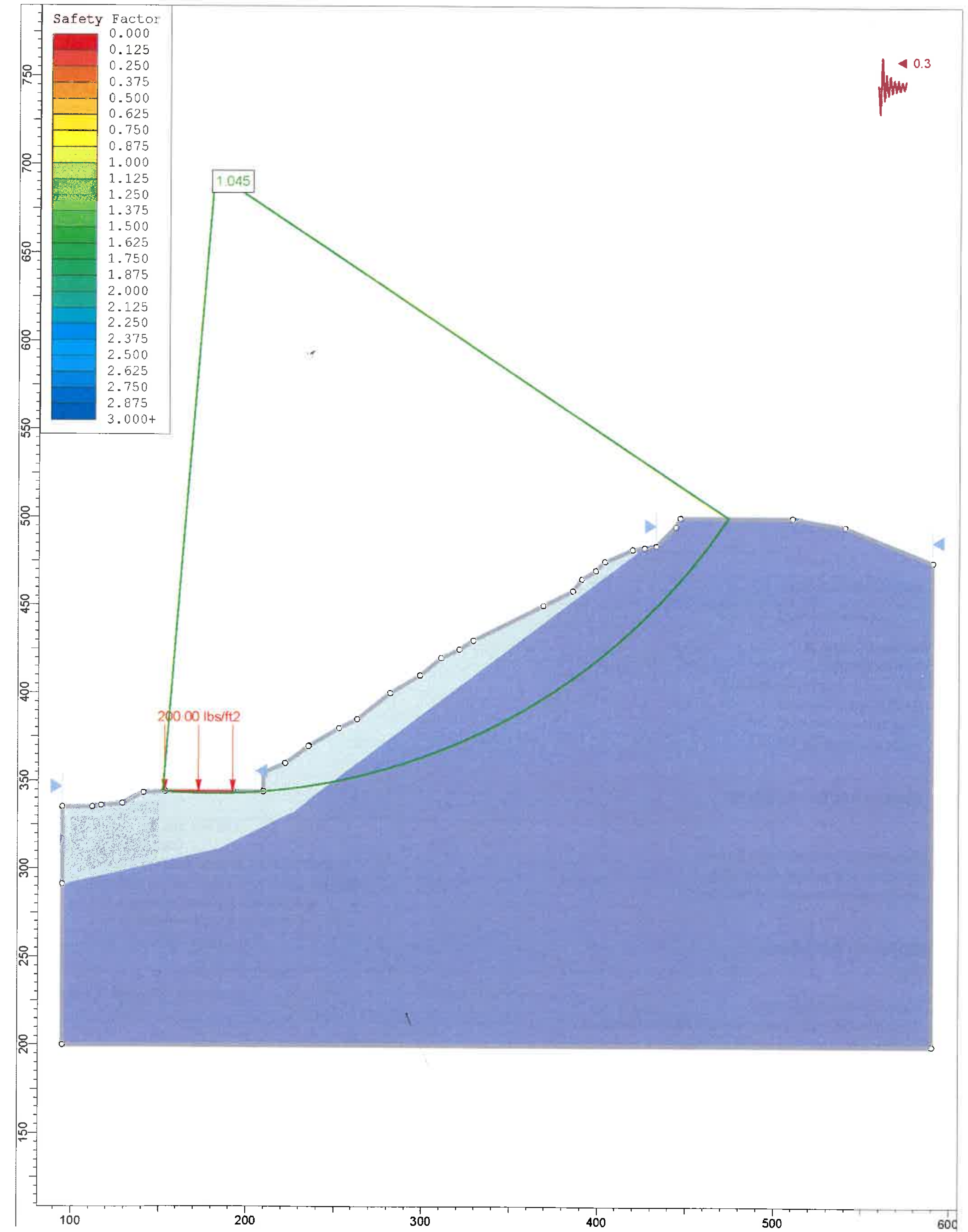
X	Y
153.275	344.081
140.964	343.238
128.856	337.041
116.896	335.991
111.88	335.097
94.8731	335.084
94.8731	291.084
95	200
590	200
590	475
540	495
510	500
446.251	500.152
443.669	495.103
432.601	484.152

426.018	483.101
419.158	482.006
403.414	475.105
398.385	470.058
390.397	465.107
385.381	458.546
368.559	450.108
328.686	430.106
320.747	425.106
310.455	420.056
298.376	410.104
281.378	400.104
262.504	385.06
252.324	380.109
235.506	370.149
235	369.777
221.82	360.1
209.339	354.885
209.339	344.082
192.047	344.082

Material Boundary

X	Y
94.8731	335.084
94.873	291.084
94.8731	291.084
185.873	312
225.873	332
235	338.891
426.018	483.101

Z-Z' Pseudo-Static Slope Stability



November 18, 2014
Project 2793

Slide Analysis Information

2793 Z-Z' Pseudo-Static Slope Stability Analysis

Project Summary

File Name: 2793 Z-Z'
Slide Modeler Version: 6.031
Project Title: 2793 Z-Z' Pseudo-Static Slope Stability Analysis
Date Created: 11/17/2014, 10:17:37 AM

General Settings

Units of Measurement: Imperial Units
Time Units: days
Permeability Units: feet/second
Failure Direction: Right to Left
Data Output: Standard
Maximum Material Properties: 20
Maximum Support Properties: 20

Analysis Options

Analysis Methods Used

Bishop simplified

Number of slices: 25
Tolerance: 0.005
Maximum number of iterations: 50
Check malpha < 0.2: Yes
Initial trial value of FS: 1
Steffensen Iteration: Yes

Groundwater Analysis

Groundwater Method: Water Surfaces
Pore Fluid Unit Weight: 62.4 lbs/ft3
Advanced Groundwater Method: None

Random Numbers

Pseudo-random Seed: 10116
Random Number Generation Method: Park and Miller v.3

Surface Options

Surface Type: Circular
Search Method: Slope Search

Slide Analysis Information

2793 Z-Z' Pseudo-Static Slope Stability Analysis

Project Summary

File Name: 2793 Z-Z'
Slide Modeler Version: 6.031
Project Title: 2793 Z-Z' Pseudo-Static Slope Stability Analysis
Date Created: 11/17/2014, 10:17:37 AM

General Settings

Units of Measurement: Imperial Units
Time Units: days
Permeability Units: feet/second
Failure Direction: Right to Left
Data Output: Standard
Maximum Material Properties: 20
Maximum Support Properties: 20

Analysis Options

Analysis Methods Used

Bishop simplified

Number of slices: 25
Tolerance: 0.005
Maximum number of iterations: 50
Check malpha < 0.2: Yes
Initial trial value of FS: 1
Steffensen Iteration: Yes

Groundwater Analysis

Groundwater Method: Water Surfaces
Pore Fluid Unit Weight: 62.4 lbs/ft3
Advanced Groundwater Method: None

Random Numbers

Pseudo-random Seed: 10116
Random Number Generation Method: Park and Miller v.3

Surface Options

Surface Type: Circular
Search Method: Slope Search

Number of Surfaces: 5000
Upper Angle: Not Defined
Lower Angle: Not Defined
Composite Surfaces: Disabled
Reverse Curvature: Create Tension Crack
Minimum Elevation: Not Defined
Minimum Depth: Not Defined

Loading

Seismic Load Coefficient (Horizontal): 0.3
1 Distributed Load present

Distributed Load 1

Distribution: Constant
Magnitude [psf]: 200
Orientation: Normal to boundary

Material Properties

Property	BEDROCK	FAN DEPOSITS
Color		
Strength Type	Mohr-Coulomb	Mohr-Coulomb
Unit Weight [lbs/ft3]	145	135
Cohesion [psf]	1000	250
Friction Angle [deg]	35	33
Water Surface	None	None
Ru Value	0	0

Global Minimums

Method: bishop simplified

FS: 1.045080
Center: 180.804, 694.667
Radius: 351.807
Left Slip Surface Endpoint: 152.315, 344.015
Right Slip Surface Endpoint: 473.902, 500.086
Resisting Moment=3.89944e+008 lb-ft
Driving Moment=3.73123e+008 lb-ft
Total Slice Area=10565.3 ft2

Valid / Invalid Surfaces

Method: bishop simplified

Number of Valid Surfaces: 4909
Number of Invalid Surfaces: 91

Error Codes:

Error Code -113 reported for 91 surfaces

Error Codes

The following errors were encountered during the computation:

-113 = Surface intersects outside slope limits.

Slice Data

Global Minimum Query (bishop simplified) - Safety Factor: 1.04508

Slice Number	Width [ft]	Weight [lbs]	Base Material	Base Cohesion [psf]	Base Friction Angle [degrees]	Shear Stress [psf]	Shear Strength [psf]	Base Normal Stress [psf]	Pore Pressure [psf]	Effective Normal Stress [psf]
1	12.102	735.12	FAN DEPOSITS	250	33	407.573	425.946	270.933	0	270.933
2	12.102	1662.22	FAN DEPOSITS	250	33	457.195	477.805	350.789	0	350.789
3	12.102	1903.54	FAN DEPOSITS	250	33	459.801	480.529	354.983	0	354.983
4	12.102	1464.19	FAN DEPOSITS	250	33	341.212	356.594	164.14	0	164.14
5	12.102	5769.61	FAN DEPOSITS	250	33	511.919	534.996	438.855	0	438.855
6	12.102	22901.4	FAN DEPOSITS	250	33	1325.46	1385.21	1748.08	0	1748.08
7	12.102	33167.3	FAN DEPOSITS	250	33	1782.62	1862.98	2483.78	0	2483.78
8	12.102	42746.3	FAN DEPOSITS	250	33	2189.39	2288.09	3138.38	0	3138.38
9	13.2218	55321.6	BEDROCK	1000	35	3280.94	3428.85	3468.76	0	3468.76
10	13.2218	65731	BEDROCK	1000	35	3653.95	3818.67	4025.48	0	4025.48
11	13.2218	77989.3	BEDROCK	1000	35	4085.12	4269.28	4669.02	0	4669.02
12	13.2218	85695.4	BEDROCK	1000	35	4305.38	4499.47	4997.76	0	4997.76
13	13.2218	95172.1	BEDROCK	1000	35	4582.17	4788.73	5410.86	0	5410.86
14	13.2218	99740.1	BEDROCK	1000	35	4648.02	4857.55	5509.16	0	5509.16
15	13.2218	102443	BEDROCK	1000	35	4633.21	4842.07	5487.05	0	5487.05
16	13.2218	102553	BEDROCK	1000	35	4515.47	4719.03	5311.32	0	5311.32
17	13.2218	101258	BEDROCK	1000	35	4344.83	4540.69	5056.63	0	5056.63
18	13.2218	98449.8	BEDROCK	1000	35	4120.98	4306.75	4722.54	0	4722.54
19	13.2218	97781.3	BEDROCK	1000	35	3972.29	4151.36	4500.61	0	4500.61
20	13.2218	98779.8	BEDROCK	1000	35	3875.15	4049.84	4355.62	0	4355.62
21	13.2218	91534.7	BEDROCK	1000	35	3507.47	3665.59	3806.85	0	3806.85
22	13.2218	75262.9	BEDROCK	1000	35	2873.87	3003.42	2861.18	0	2861.18
23	13.2218	66732.1	BEDROCK	1000	35	2499.31	2611.98	2302.14	0	2302.14
24	13.2218	51359.7	BEDROCK	1000	35	1948.08	2035.9	1479.42	0	1479.42
25	13.2218	17840.4	BEDROCK	1000	35	958.53	1001.74	2.48579	0	2.48579

Interslice Data

Global Minimum Query (bishop simplified) - Safety Factor: 1.04508

Slice Number	X coordinate [ft]	Y coordinate - Bottom [ft]	Interslice Normal Force [lbs]	Interslice Shear Force [lbs]	Interslice Force Angle [degrees]
1	152.315	344.015	0	0	0
2	164.417	343.242	4921.02	0	0
3	176.519	342.886	10079.6	0	0
4	188.621	342.947	15051	0	0

Error Code -113 reported for 91 surfaces

Error Codes

The following errors were encountered during the computation:

-113 = Surface intersects outside slope limits.

Slice Data

Global Minimum Query (bishop simplified) - Safety Factor: 1.04508

Slice Number	Width [ft]	Weight [lbs]	Base Material	Base Cohesion [psf]	Base Friction Angle [degrees]	Shear Stress [psf]	Shear Strength [psf]	Base Normal Stress [psf]	Pore Pressure [psf]	Effective Normal Stress [psf]
1	12.102	735.12	FAN DEPOSITS	250	33	407.573	425.946	270.933	0	270.933
2	12.102	1662.22	FAN DEPOSITS	250	33	457.195	477.805	350.789	0	350.789
3	12.102	1903.54	FAN DEPOSITS	250	33	459.801	480.529	354.983	0	354.983
4	12.102	1464.19	FAN DEPOSITS	250	33	341.212	356.594	164.14	0	164.14
5	12.102	5769.61	FAN DEPOSITS	250	33	511.919	534.996	438.855	0	438.855
6	12.102	22901.4	FAN DEPOSITS	250	33	1325.46	1385.21	1748.08	0	1748.08
7	12.102	33167.3	FAN DEPOSITS	250	33	1782.62	1862.98	2483.78	0	2483.78
8	12.102	42746.3	FAN DEPOSITS	250	33	2189.39	2288.09	3138.38	0	3138.38
9	13.2218	55321.6	BEDROCK	1000	35	3280.94	3428.85	3468.76	0	3468.76
10	13.2218	65731	BEDROCK	1000	35	3653.95	3818.67	4025.48	0	4025.48
11	13.2218	77989.3	BEDROCK	1000	35	4085.12	4269.28	4669.02	0	4669.02
12	13.2218	85695.4	BEDROCK	1000	35	4305.38	4499.47	4997.76	0	4997.76
13	13.2218	95172.1	BEDROCK	1000	35	4582.17	4788.73	5410.86	0	5410.86
14	13.2218	99740.1	BEDROCK	1000	35	4648.02	4857.55	5509.16	0	5509.16
15	13.2218	102443	BEDROCK	1000	35	4633.21	4842.07	5487.05	0	5487.05
16	13.2218	102553	BEDROCK	1000	35	4515.47	4719.03	5311.32	0	5311.32
17	13.2218	101258	BEDROCK	1000	35	4344.83	4540.69	5056.63	0	5056.63
18	13.2218	98449.8	BEDROCK	1000	35	4120.98	4306.75	4722.54	0	4722.54
19	13.2218	97781.3	BEDROCK	1000	35	3972.29	4151.36	4500.61	0	4500.61
20	13.2218	98779.8	BEDROCK	1000	35	3875.15	4049.84	4355.62	0	4355.62
21	13.2218	91534.7	BEDROCK	1000	35	3507.47	3665.59	3806.85	0	3806.85
22	13.2218	75262.9	BEDROCK	1000	35	2873.87	3003.42	2861.18	0	2861.18
23	13.2218	66732.1	BEDROCK	1000	35	2499.31	2611.98	2302.14	0	2302.14
24	13.2218	51359.7	BEDROCK	1000	35	1948.08	2035.9	1479.42	0	1479.42
25	13.2218	17840.4	BEDROCK	1000	35	958.53	1001.74	2.48579	0	2.48579

Interslice Data

Global Minimum Query (bishop simplified) - Safety Factor: 1.04508

Slice Number	X coordinate [ft]	Y coordinate - Bottom [ft]	Interslice Normal Force [lbs]	Interslice Shear Force [lbs]	Interslice Force Angle [degrees]
1	152.315	344.015	0	0	0
2	164.417	343.242	4921.02	0	0
3	176.519	342.886	10079.6	0	0
4	188.621	342.947	15051	0	0

5	200.723	343.424	18662.3	0	0
6	212.825	344.32	22732.8	0	0
7	224.927	345.638	29598.3	0	0
8	237.029	347.382	36887.4	0	0
9	249.131	349.559	43724.6	0	0
10	262.353	352.442	60503.3	0	0
11	275.575	355.865	75311.3	0	0
12	288.796	359.845	87340.1	0	0
13	302.018	364.401	95778.2	0	0
14	315.24	369.559	99898	0	0
15	328.462	375.347	99539.6	0	0
16	341.684	381.8	94652.7	0	0
17	354.906	388.96	85553.9	0	0
18	368.127	396.878	72577.1	0	0
19	381.349	405.617	56253.8	0	0
20	394.571	415.254	36065.3	0	0
21	407.793	425.884	11360.5	0	0
22	421.015	437.632	-14451.6	0	0
23	434.236	450.658	-36307.7	0	0
24	447.458	465.181	-56718.9	0	0
25	460.68	481.506	-70523.5	0	0
26	473.902	500.086	0	0	0

List Of Coordinates

Distributed Load

X	Y
192.047	344.082
153.275	344.081

External Boundary

X	Y
153.275	344.081
140.964	343.238
128.856	337.041
116.896	335.991
111.88	335.097
94.8731	335.084
94.8731	291.084
95	200
590	200
590	475
540	495
510	500
446.251	500.152
443.669	495.103
432.601	484.152

426.018	483.101
419.158	482.006
403.414	475.105
398.385	470.058
390.397	465.107
385.381	458.546
368.559	450.108
328.686	430.106
320.747	425.106
310.455	420.056
298.376	410.104
281.378	400.104
262.504	385.06
252.324	380.109
235.506	370.149
235	369.777
221.82	360.1
209.339	354.885
209.339	344.082
192.047	344.082

Material Boundary

X	Y
94.8731	335.084
94.873	291.084
94.8731	291.084
185.873	312
225.873	**332**
235	338.891
426.018	483.101

426.018	483.101
419.158	482.006
403.414	475.105
398.385	470.058
390.397	465.107
385.381	458.546
368.559	450.108
328.686	430.106
320.747	425.106
310.455	420.056
298.376	410.104
281.378	400.104
262.504	385.06
252.324	380.109
235.506	370.149
235	369.777
221.82	360.1
209.339	354.885
209.339	344.082
192.047	344.082

Material Boundary

X	Y
94.8731	335.084
94.873	291.084
94.8731	291.084
185.873	312
225.873	332
235	338.891
426.018	483.101

SCREENING ACCELERATION

CALCULATE THE PEAK HORIZONTAL ACCELERATION DUE TO SEISMIC LOADINGS. (REFERENCE: Abrahamson, N.A. and Silva, W.L. (1996), Empirical Ground Motion Models,report prepared for Brookhaven National Laboratory, New York, NY, May, 144 pp.).

CALCULATION PARAMETERS OBTAINED FROM USGS SEISMIC DESIGN MAPS AND INTERACTIVE DEAGGREGATION BASED ON 2010 ASCE 7 WITH MARCH 2013 ERRATA

SEISMIC PARAMETERS

SITE LATITUDE	34.251 °N	SITE SOIL CLASSIFICATION	C
SITE LONGITUDE	118.2684 °W	RISK CATEGORY	I, II, or III
SPECTRAL PERIOD	0.0 s	EXCEEDANCE PROBABILITY	10% in 50 years
		SHEAR WAVE VELOCITY (V_s)	760 m/s

CALCULATION PARAMETERS

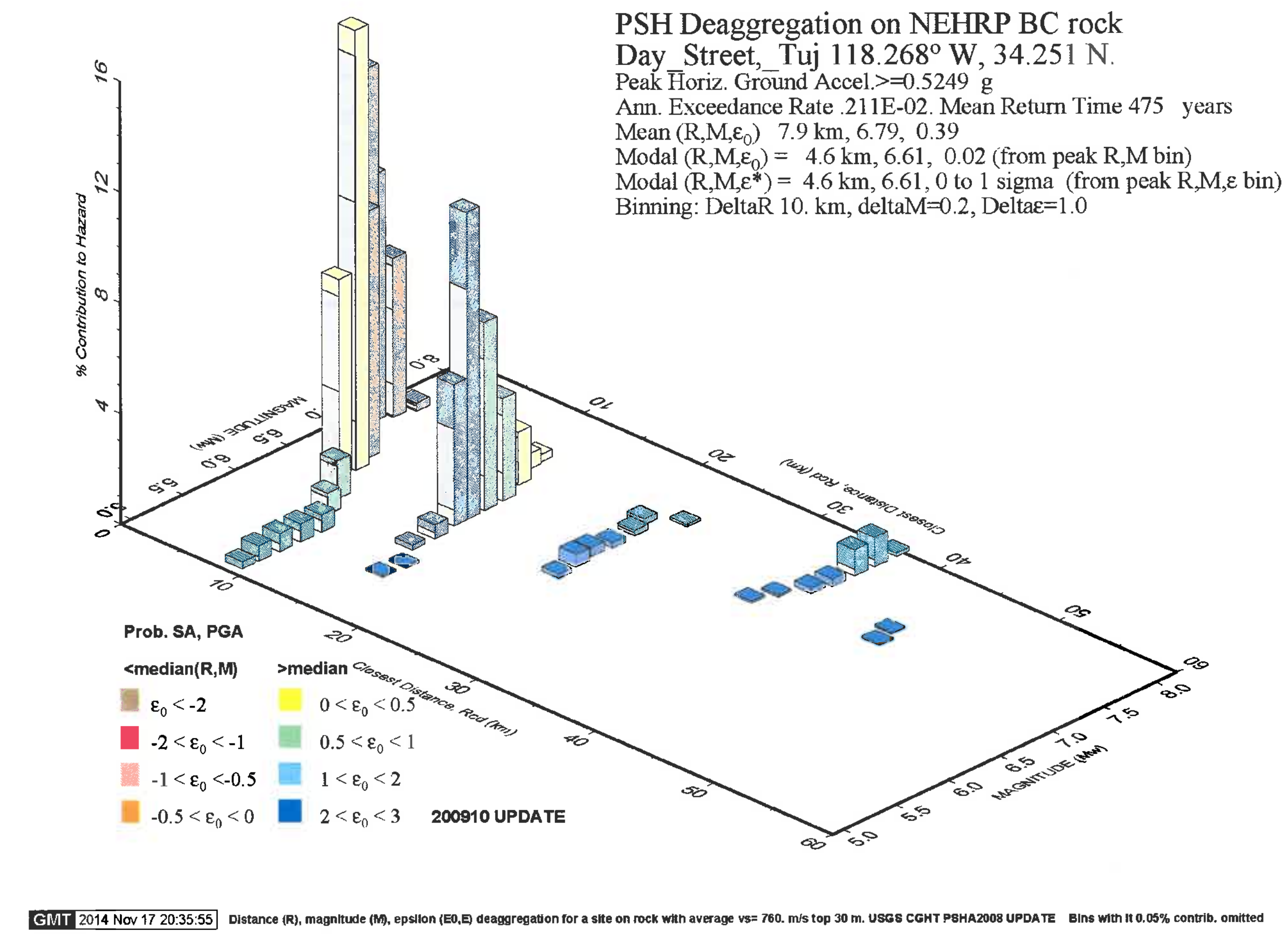
PGAm	0.996 g	DISTANCE FROM LOCATION (r)	4.6 km
MAGNITUDE (M)	6.61	DISPLACEMENT (u)	5 cm

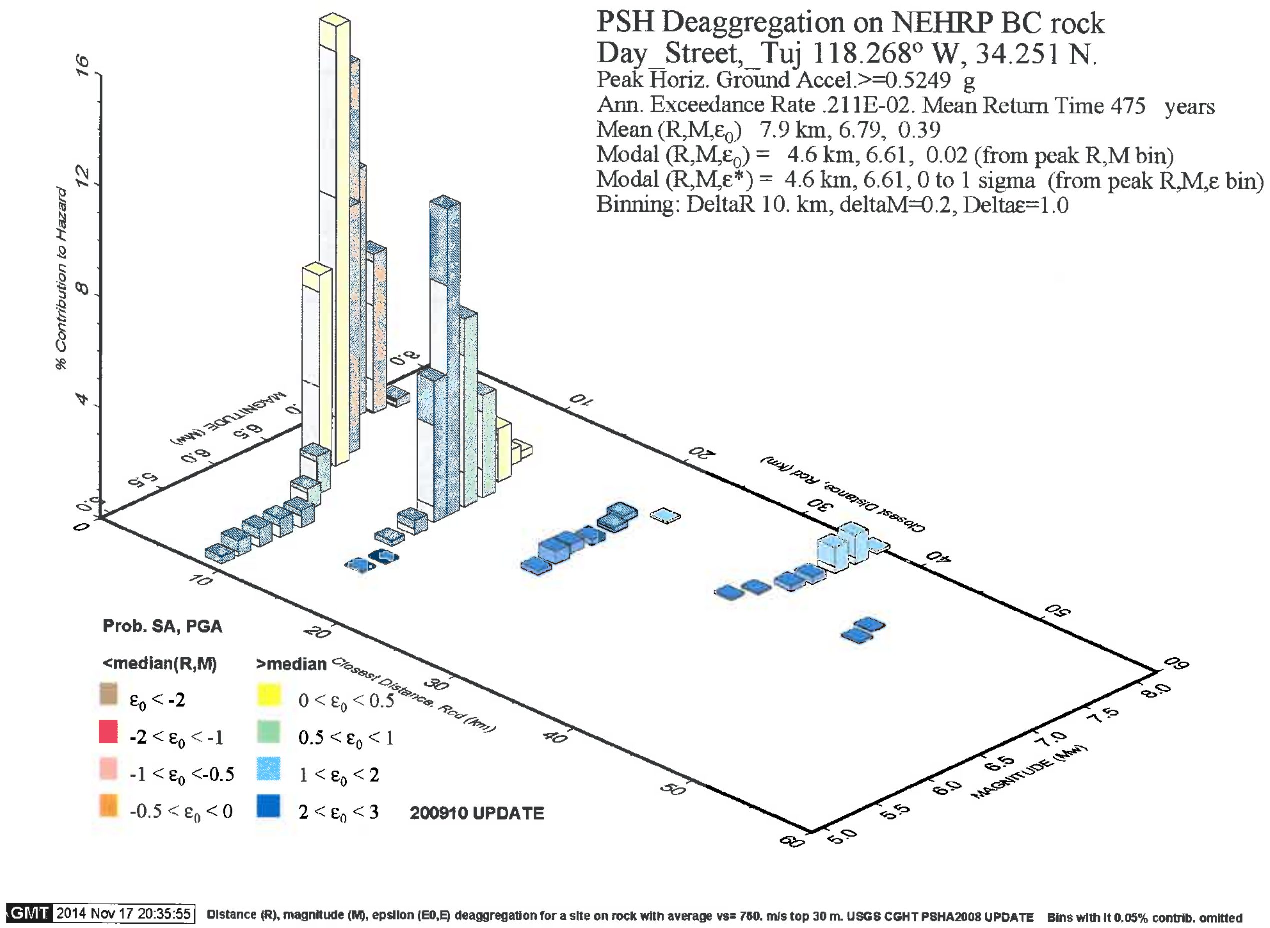
CALCULATED RESULTS

PGA	0.66 g
$D_{5\text{-}95}$	10.16 s
NRF	0.83
f_{eq}	0.46
K_{eq}	**0.30**

CONCLUSIONS:

THE CALCULATED HORIZONTAL ACCELERATION DUE TO SEISMIC FORCES, ARE SHOWN IN THE TABLE.


PSH Deaggregation on NEHRP BC rock
Day_Street,_Tuj 118.268° W, 34.251 N.
Peak Horiz. Ground Accel.>=0.5249 g
Ann. Exceedance Rate .211E-02. Mean Return Time 475 years
Mean (R,M,ε0) 7.9 km, 6.79, 0.39
Modal (R,M,ε0) = 4.6 km, 6.61, 0.02 (from peak R,M bin)
Modal (R,M,ε*) = 4.6 km, 6.61, 0 to 1 sigma (from peak R,M,ε bin)
Binning: DeltaR 10. km, deltaM=0.2, Deltaε=1.0
% Contribution to Hazard
Closest Distance, Rcd (km)
MAGNITUDE (Mw)
Prob. SA, PGA
<median(R,M)
>median
ε0 < -2
-2 < ε0 < -1
-1 < ε0 <-0.5
-0.5 < ε0 < 0
0 < ε0 < 0.5
0.5 < ε0 < 1
1 < ε0 < 2
2 < ε0 < 3
200910 UPDATE
GMT 2014 Nov 17 20:35:55
Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on rock with average vs= 760. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted



USGS Design Maps Detailed Report

ASCE 7-10 Standard (34.251°N, 118.2684°W)

Site Class C – "Very Dense Soil and Soft Rock", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1] S_S = 2.666 g

From Figure 22-2 [2] S_1 = 0.971 g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class C, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index PI > 20, • Moisture content $w \ge 40\%$, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4-1: Site Coefficient F_a

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	$\boxed{1.0}$
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_S

For Site Class = C and S_S = 2.666 g, F_a = 1.000

Table 11.4-2: Site Coefficient F_v

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	$\boxed{1.3}$
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = C and S_1 = 0.971 g, F_v = 1.300

Section 11.4.3 — Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = C and S_S = 2.666 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_V

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = C and S_1 = 0.971 g, F_v = 1.300

Equation (11.4–1): $S_{MS} = F_aS_S = 1.000 \times 2.666 = 2.666$ g

Equation (11.4–2): $S_{M1} = F_vS_1 = 1.300 \times 0.971 = 1.263$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

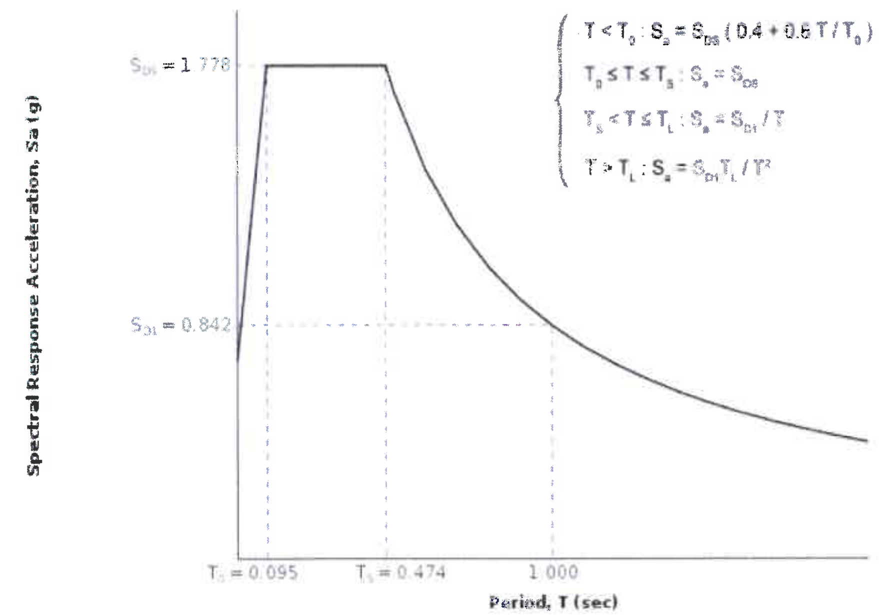
Equation (11.4–3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 2.666 = 1.778$ g

Equation (11.4–4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 1.263 = 0.842$ g

Section 11.4.5 — Design Response Spectrum

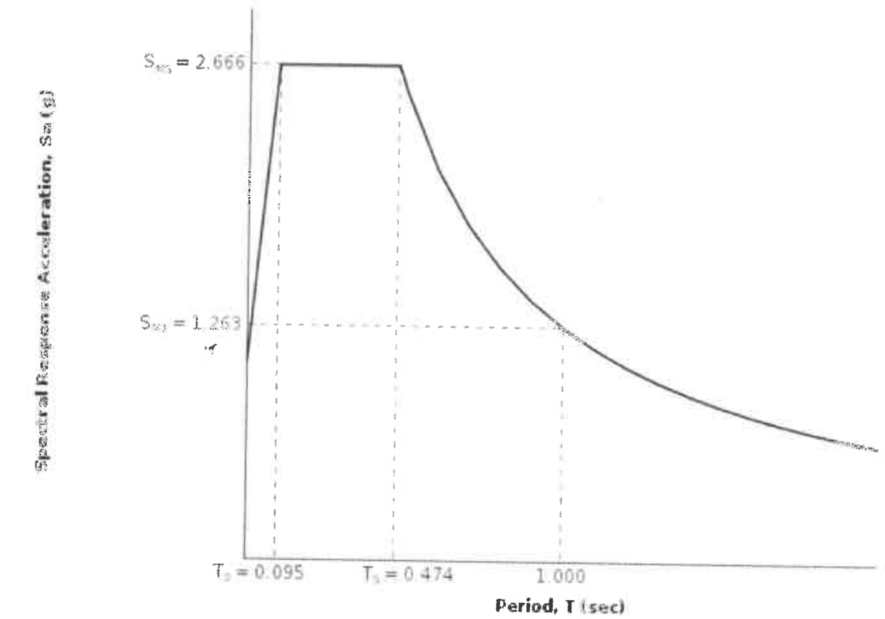
From Figure 22-12 [3] T_L = 8 seconds

Figure 11.4–1: Design Response Spectrum



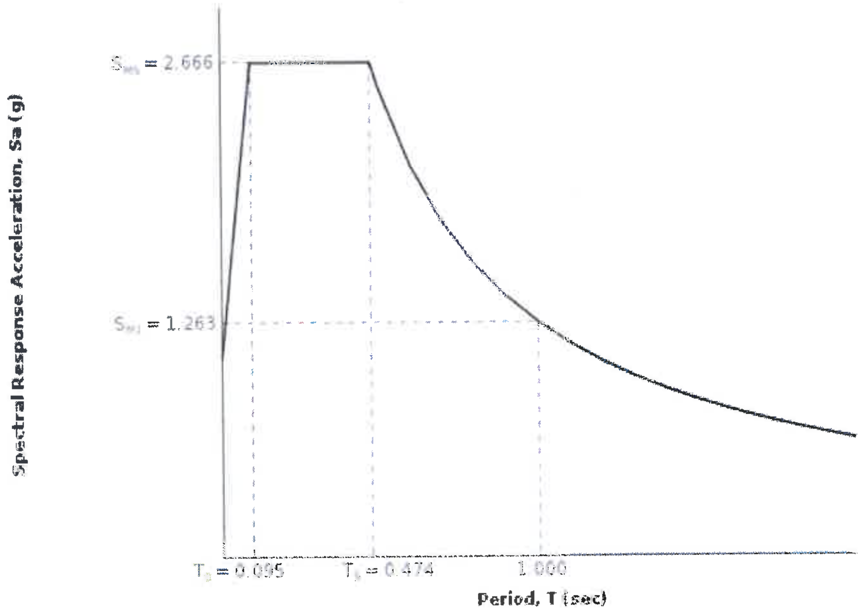
Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.996

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.996 = 0.996$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of PGA

For Site Class = C and PGA = 0.996 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 0.955

From Figure 22-18 [6] C_{R1} = 0.950

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Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 1.778 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.842 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = E

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 1.778 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.842 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = E

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

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2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

RETAINING WALL

CALCULATE THE DESIGN MINIMUM EQUIVALENT FLUID PRESSURE (EFP) FOR PROPOSED RETAINING WALLS. THE WALL HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. THE MONONOBE-OKABE METHOD USED TO CALCULATE SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	Fan Deposits	WALL HEIGHT	8 feet
SHEAR DIAGRAM:	0	BACKSLOPE ANGLE:	36 degrees
COHESION:	250 psf	SURCHARGE:	0 pounds
PHI ANGLE:	33 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	135 pcf	INITIAL FAILURE ANGLE:	40 degrees
SAFETY FACTOR:	1.5	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	0 degrees	INITIAL TENSION CRACK:	5 feet
CD (C/FS):	166.7 psf	FINAL TENSION CRACK:	40 feet

PHID = ATAN(TAN(PHI)/FS) = 23.4 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	40 degrees
AREA OF TRIAL FAILURE WEDGE	164.8 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	22251.5 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	32.6 feet
DEPTH OF TENSION CRACK	5.2 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	25.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	1421.1 pounds
CALCULATED EQUIVALENT FLUID PRESSURE	44.4 pcf

RETAINING WALL

CALCULATE THE DESIGN MINIMUM EQUIVALENT FLUID PRESSURE (EFP) FOR PROPOSED RETAINING WALLS. THE WALL HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. THE MONONOBE-OKABE METHOD USED TO CALCULATE SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	Fan Deposits	WALL HEIGHT	8 feet
SHEAR DIAGRAM:	0	BACKSLOPE ANGLE:	36 degrees
COHESION:	250 psf	SURCHARGE:	0 pounds
PHI ANGLE:	33 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	135 pcf	INITIAL FAILURE ANGLE:	40 degrees
SAFETY FACTOR:	1	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	0 degrees	INITIAL TENSION CRACK:	5 feet
CD (C/FS):	250.0 psf	FINAL TENSION CRACK:	40 feet

PHID = ATAN(TAN(PHI)/FS) = 33.0 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0.3 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	40 degrees
AREA OF TRIAL FAILURE WEDGE	179.9 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	24283.5 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	36.6 feet
DEPTH OF TENSION CRACK	4.8 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	28.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	2545.5 pounds

RETAINING WALL

CALCULATE THE DESIGN MINIMUM EQUIVALENT FLUID PRESSURE (EFP) FOR PROPOSED RETAINING WALLS. THE WALL HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. THE MONONOBE-OKABE METHOD USED TO CALCULATE SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	Fan Deposits	WALL HEIGHT	8 feet
SHEAR DIAGRAM:	0	BACKSLOPE ANGLE:	36 degrees
COHESION:	250 psf	SURCHARGE:	0 pounds
PHI ANGLE:	33 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	135 pcf	INITIAL FAILURE ANGLE:	40 degrees
SAFETY FACTOR:	1	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	0 degrees	INITIAL TENSION CRACK:	5 feet
CD (C/FS):	250.0 psf	FINAL TENSION CRACK:	40 feet

PHID = ATAN(TAN(PHI)/FS) = 33.0 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0.3 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	40 degrees
AREA OF TRIAL FAILURE WEDGE	179.9 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	24283.5 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	36.6 feet
DEPTH OF TENSION CRACK	4.8 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	28.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	2545.5 pounds

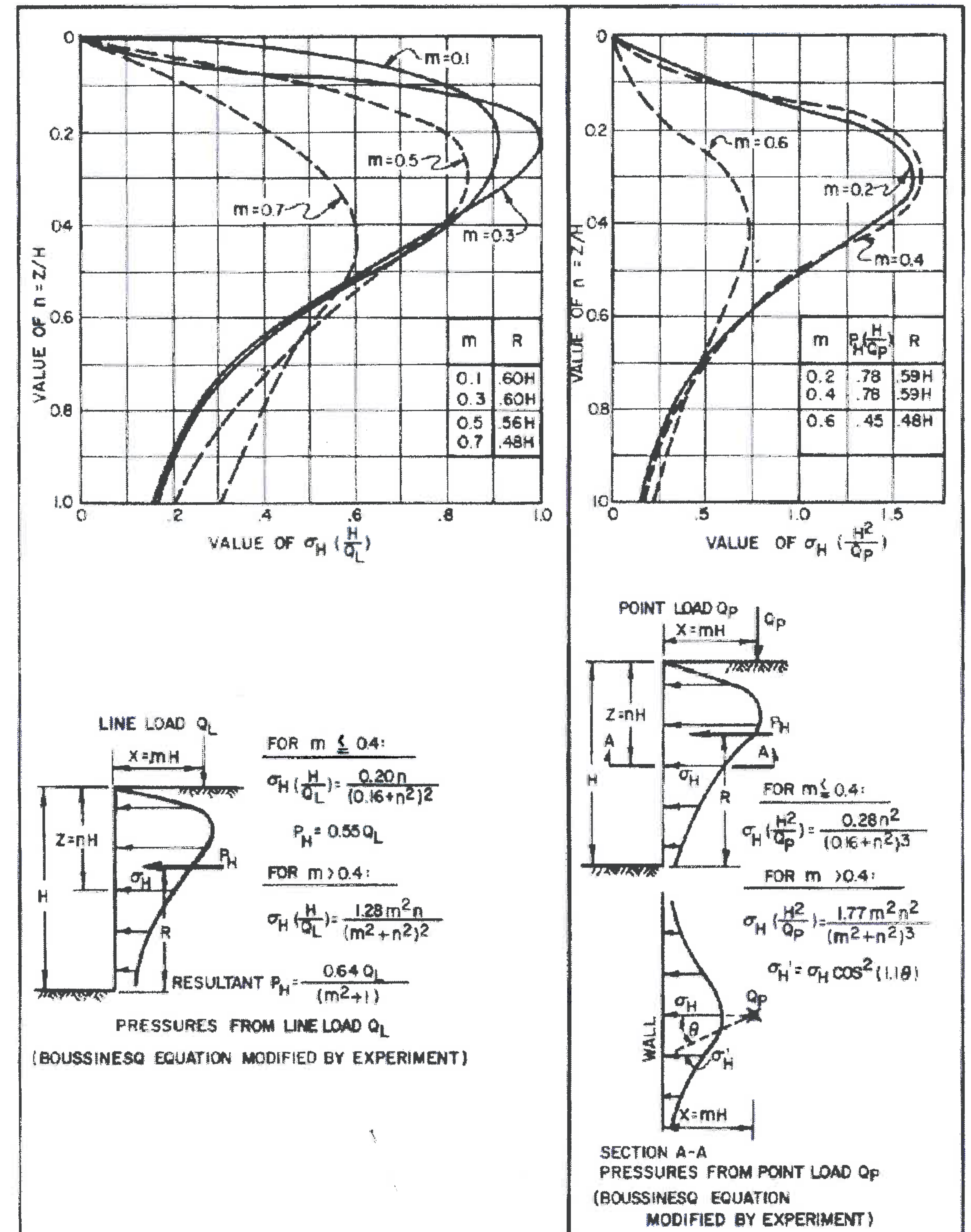

FIGURE 11
Horizontal Pressures on Rigid Wall from Surface Load

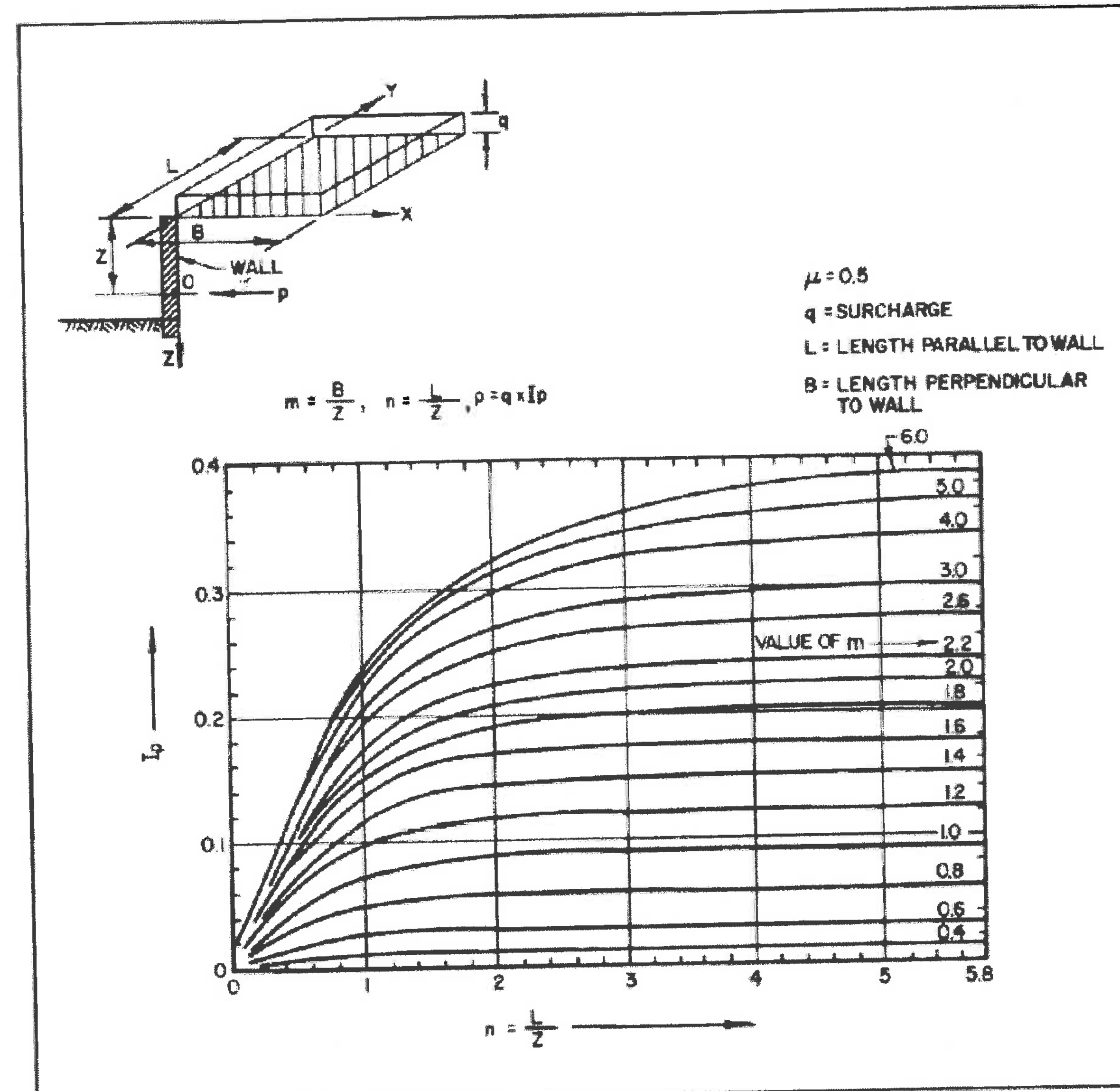

FIGURE 12
Lateral Pressure on an Unyielding Wall due to
Uniform Rectangular Surface Load

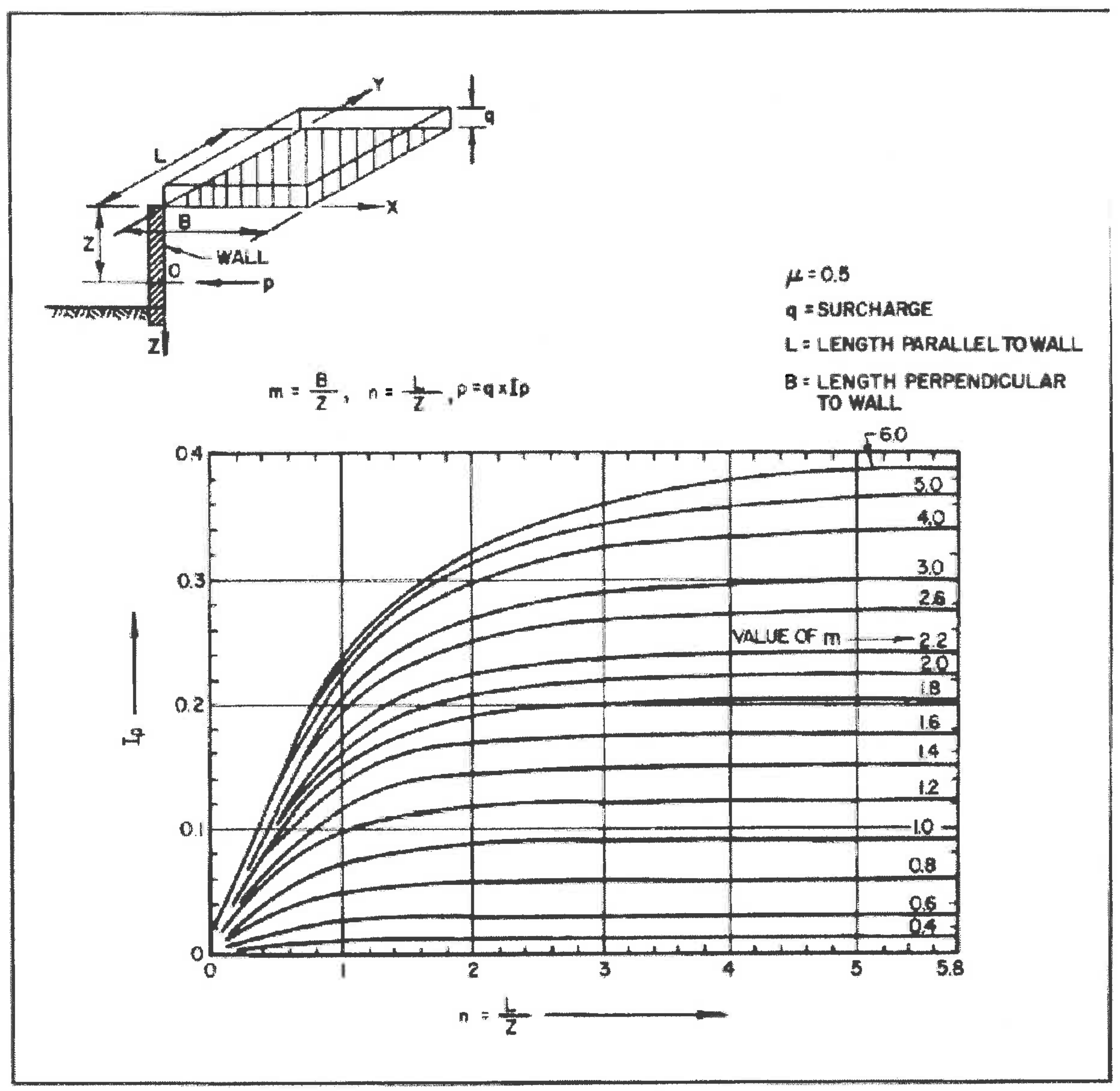

FIGURE 12
Lateral Pressure on an Unyielding Wall due to Uniform Rectangular Surface Load

APPENDIX IV

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