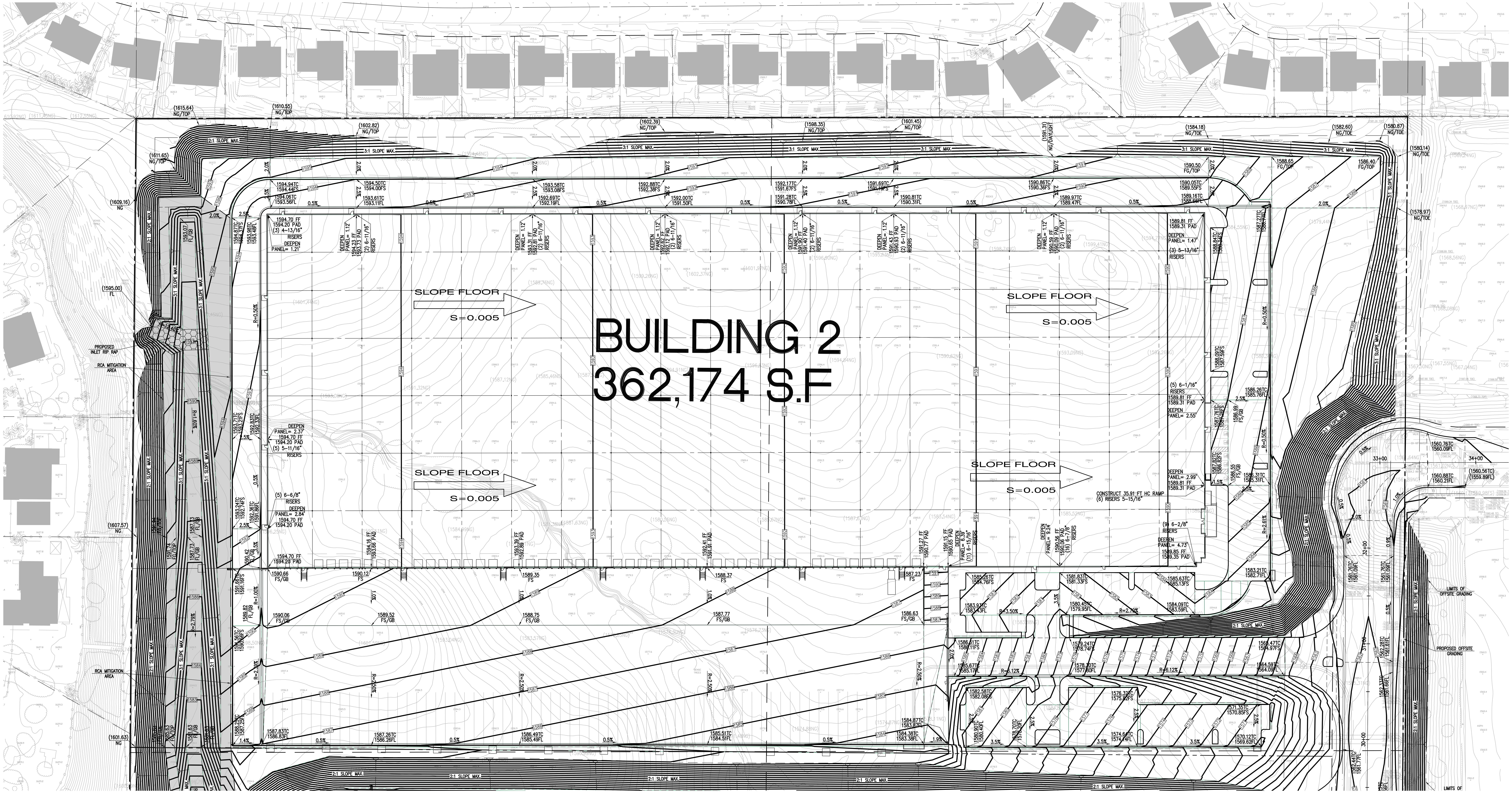


FOR BUILDING 1 - SEE SHEETS 1-3

0 20 40 80 120

SCALE: 1"=40'

Last Update: 3/29/16

PREPARED FOR:

Hillwood Investments
901 Via Piemonte, Suite 175
Ontario, CA 91764
PHONE: (909)382-0033
FAX: (909)382-0073

Thienes Engineering, Inc.
CIVIL ENGINEERING • LAND SURVEYING
14349 FIRESTONE BOULEVARD
LA MIRADA, CALIFORNIA 90638
PH.(714)521-4811 FAX(714)521-4173

CITY OF RIVERSIDE
PUBLIC WORKS DEPARTMENT

CONCEPTUAL GRADING PLAN

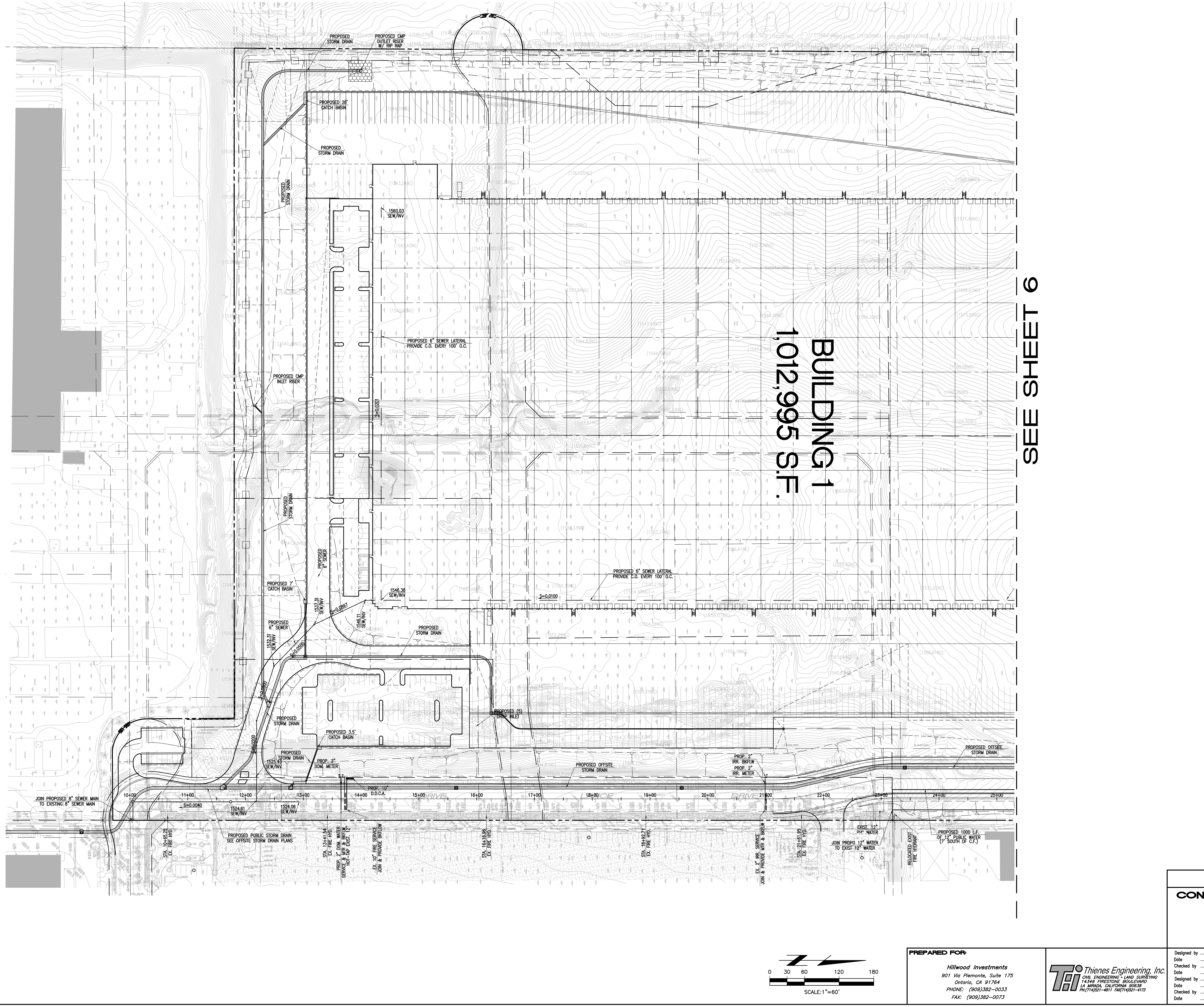
SYCAMORE V

LANCE DRIVE
BUILDING 2

Designed by ______ Date
Checked by ______ Date
Designed by ______ Date
Checked by ______ Date

Approved by ______ Date

Public Works Director ______ R.C.E. XXXXX



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CITY OF RIVERSIDE
PUBLIC WORKS DEPARTMENT

CONCEPTUAL UTILITY PLAN
SYCAMORE V

LANCE DRIVE
BUILDING 1 AND 2

PREPARED FOR:

Hillwood Investments
901 Via Piemonte, Suite 175
Ontario, CA 91764
PHONE: (909)382-0033
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Thienes Engineering, Inc.
CIVIL ENGINEERING • LAND SURVEYING
14349 FIRESTONE BOULEVARD
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PH.(714)521-4811 FAX(714)521-4173

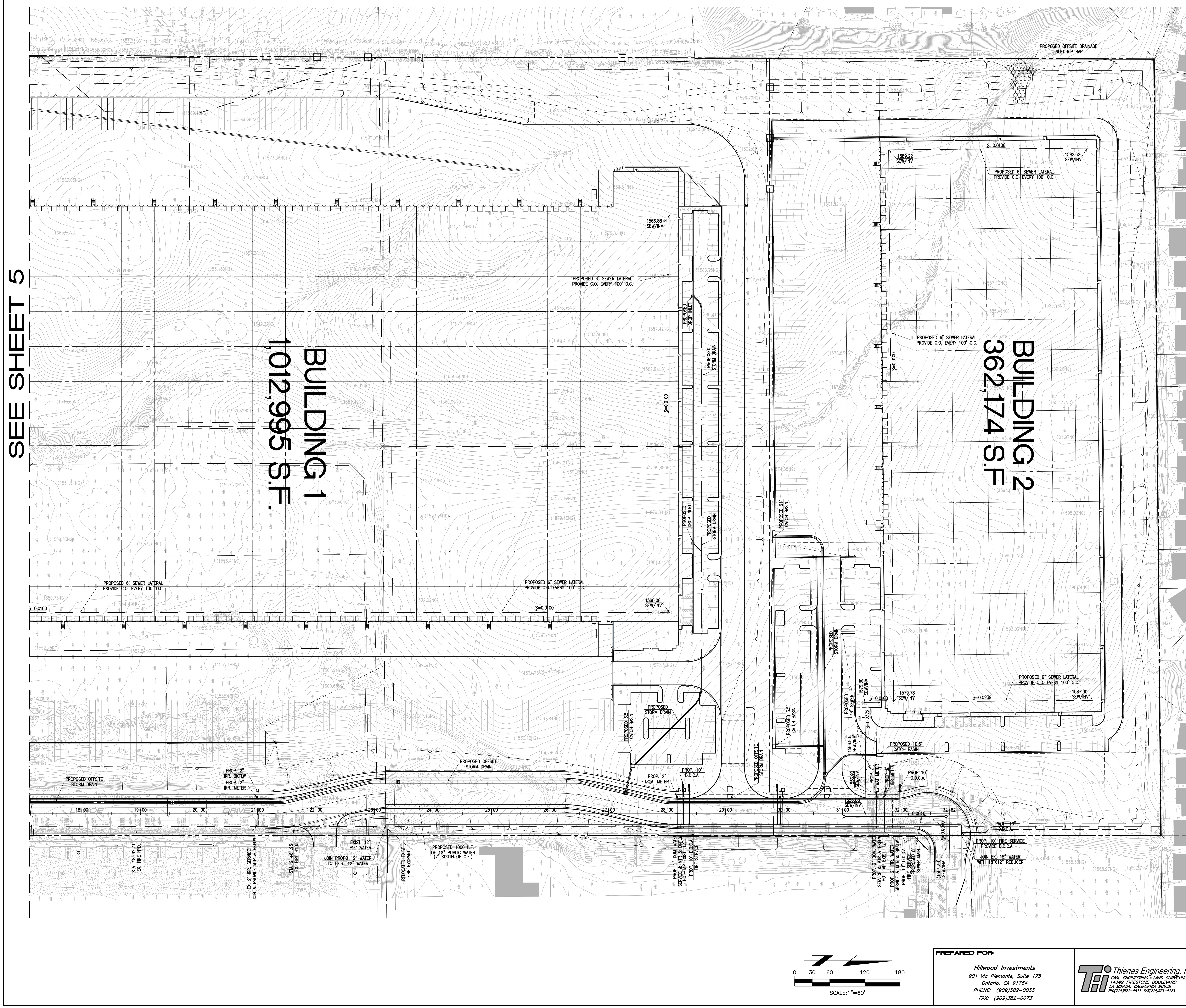
Designed by ______
Date ______
Checked by ______
Date ______
Designed by ______
Date ______
Checked by ______
Date ______

Approved by Date ______

Public Works Director R.C.E. XXXXX

Sheet 5 of 6 Sheets

3261/5 OF 6 SHEET



Last Update: 3/29/16
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CITY OF RIVERSIDE
PUBLIC WORKS DEPARTMENT

CONCEPTUAL UTILITY PLAN

SYCAMORE V

LANCE DRIVE
BUILDING 1 AND 2

PREPARED FOR:

Hillwood Investments
901 Via Piemonte, Suite 175
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PH.(714)521-4811 FAX(714)521-4173

Designed by ______	Approved by ______ Date ______
Date ______	
Checked by ______	
Date ______	
Designed by ______	Public Works Director R.C.E. XXXXX
Date ______	
Checked by ______	
Date ______	Sheet 6 of 6 Sheets

3261/6 OF 6 SHEET

Appendix 3: Soils Information

Geotechnical Study and Other Infiltration Testing Data

C.H.J. Incorporated
1355 E. Cooley Drive, Colton, CA 92324 ◆ Phone (909) 824-7210 ◆ Fax (909) 824-7209
15345 Anacapa Road, Suite D, Victorville, CA 92392 ◆ Phone (760) 243-0506 ◆ Fax (760) 243-1225

July 23, 2008

The Magnon Companies
815 Marlborough Avenue, Suite 200
Riverside, California 92507
Attention: Mr. Ray Magnon

Job No. 07489-3

Subject: Grading Plan Review and Update to Geotechnical Investigation
Proposed Industrial Development - Sycamore V
Lance and Sierra Drives
Riverside, California

References: See Reference Sheet

Dear Mr. Magnon:

As requested, we have reviewed the 40-scale Precise Grading Plan prepared by GFB - Friedrich & Associates, Incorporated, dated June 2008, as it pertains to the geologic/geotechnical aspects of the development (Sycamore V). This firm conducted a geotechnical investigation on the site (July 20, 2007). It is our understanding that the subject site is to be developed with an approximately 1.1-million square feet industrial structure and related infrastructure. It is anticipated that the site will be developed with one- to two-story structures of concrete tilt-up construction, utilizing conventional spread foundations for support.

CUT/FILL TRANSITION:

Due to the topographic relief across the site and the varying depth of bedrock, it is not planned for the foundations of the proposed structure to bear entirely in undisturbed bedrock. Grading of the building pad area will entail maximum cuts on the order of 15 to 18 feet and maximum fills on the order of 12 to

16 feet, when alluvial/colluvial removals are included. Cut-fill transition lines are present in the southern and central portions of the proposed building (Enclosure 1). In order to mitigate the potential for differential settlement across the cut/fill transition area, the pad area including areas underlain by bedrock, should be subexcavated to provide no greater than 8 feet difference in the fill mat thickness within 150 feet of the cut/fill transition line. The 150-foot distance from the transition line is also shown on Enclosure 1. The attached draft cross sections illustrate the cut/fill transition, areas of fill, bedrock cut areas, and potential fill depths beneath footings in rock areas. Our geotechnical investigation recommended a minimum fill thickness of 18 inches beneath all structure footings.

Other portions of the Precise Grading Plan appear to be in compliance with our geotechnical investigation report. When available, the foundation plans should be reviewed by the geotechnical engineer.

CUT SLOPES:

Cut slopes up to approximately 30 feet in height and inclined at 2 horizontal to 1 vertical [2(h):1(v)] and flatter are proposed in the northern portion of the site. Retaining walls up to approximately 15 feet in height are proposed below the toes of some cut slopes. Cut slopes in competent bedrock may be safely constructed at inclinations as steep as 1.5(h):1(v) up to a maximum height of 30 feet. Where the tops of bedrock cut slopes intersect natural ground with associated topsoils and highly weathered rock, the upper 5 feet of these slopes should be constructed no steeper than 2(h):1(v). All cut slopes should be observed by the engineering geologist during grading. These recommendations are subject to change by the engineering geologist as dictated by conditions exposed during grading.

ROCK DISPOSAL:

Grading of the site may result in the generation of rock with dimensions greater than 12 inches. No rock greater than 12 inches in dimension may be buried or placed in fills within 10 feet of final grade. If it is necessary to dispose of such material on-site, the boulders should not be placed in site fills except in areas designated as "rock disposal areas". Such "rock disposal areas" should be non-buildable areas or areas below the depth of 10 feet.

In order to properly compact soils in "rock disposal areas", the rock should be placed in windrows, in such a manner as to avoid nesting and to allow access for compaction equipment. The oversized material should be placed in lifts, alternating with lifts of soil without oversized material. Continuous observation by the geotechnical engineer should be provided to confirm that adequate compaction is achieved in such areas. A typical rock disposal detail is included in Appendix "D" of the referenced geotechnical report.

Further recommendations may be made in the field depending on the actual conditions encountered.

GEOTECHNICAL UPDATE:

A site visit was performed on July 11, 2008. The purpose of the site visit was to observe the site as it presently exists and to determine whether any new or revised recommendations are necessary. As observed during our site visit, the site appeared to be in the same condition as it was during our original investigation in 2007. Except where included in this report, no additional or revised recommendations appear to be necessary at this time.

The seismic design parameters were determined according the 2007 California Building Code (CBC) based on the site class "C", "stiff soil", and are summarized in the table below.

2007 California Building Code - Seismic Parameters	
Mapped Spectral Acceleration Parameters	$S_s = 1.50$ and $S_1 = 0.60$
Site Coefficients	$F_a = 1.0$ and $F_v = 1.3$
Adjusted Maximum Considered Earthquake (MCE) Spectral Response Parameters	$S_{MS} = 1.50$ and $S_{M1} = 0.78$
Design Spectral Acceleration Parameters	$S_{DS} = 1.00$ and $S_{D1} = 0.52$

The corresponding value of peak ground acceleration based on the project design response spectrum is 0.40g.

CLOSURE:

We trust this information is as requested. If you should have any questions, please do not hesitate to contact this firm at your convenience.

Respectfully submitted,

C.H.J., INCORPORATED

PROFESSIONAL GEOLOGIST
JAY J. MARTIN
CERTIFIED ENGINEERING GEOLOGIST
Exp. 8-31-08
STATE OF CALIFORNIA

7/23/08

Jay J. Martin, E.G. 1529
Vice President

Allen D. Evans, G.E. 2060
Vice President

REGISTERED PROFESSIONAL ENGINEER
ALLEN D. EVANS
No. 2060
Exp. 03-31-09
GEOTECHNICAL
STATE OF CALIFORNIA

7/23/08

JJM/ADE:ndt

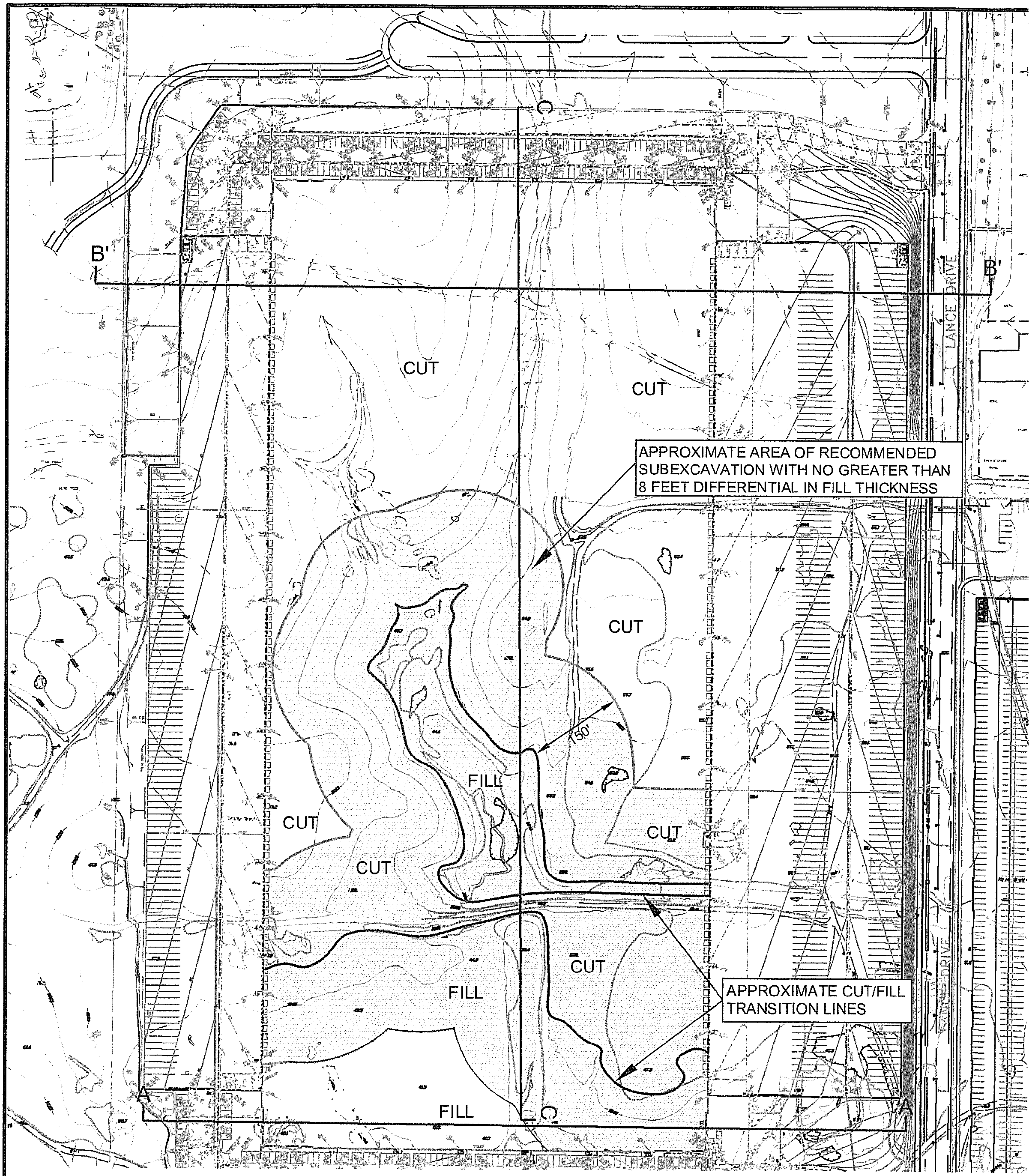
Enclosures:
1) Site Location Map
2) Cross Section A-A' (draft)
3) Cross Section B-B' (draft)
4) Cross Section C-C' (draft)

Distribution:	The Magnon Companies	(4)
	GFB - Friedrich & Associates, Inc.	(2)

REFERENCES

C.H.J, Incorporated, July 20, 2007, Geotechnical Investigation, Proposed Industrial Development, Lance and Sierra Drives, Riverside, California, Job No. 07489-3.

GFB - Friedrich & Associates, Inc., June 2008, Precise Grading Plan, Sycamore V, 6275 Lance Drive, Riverside, California.



SCALE 1"= 200'

LEGEND:

C C'

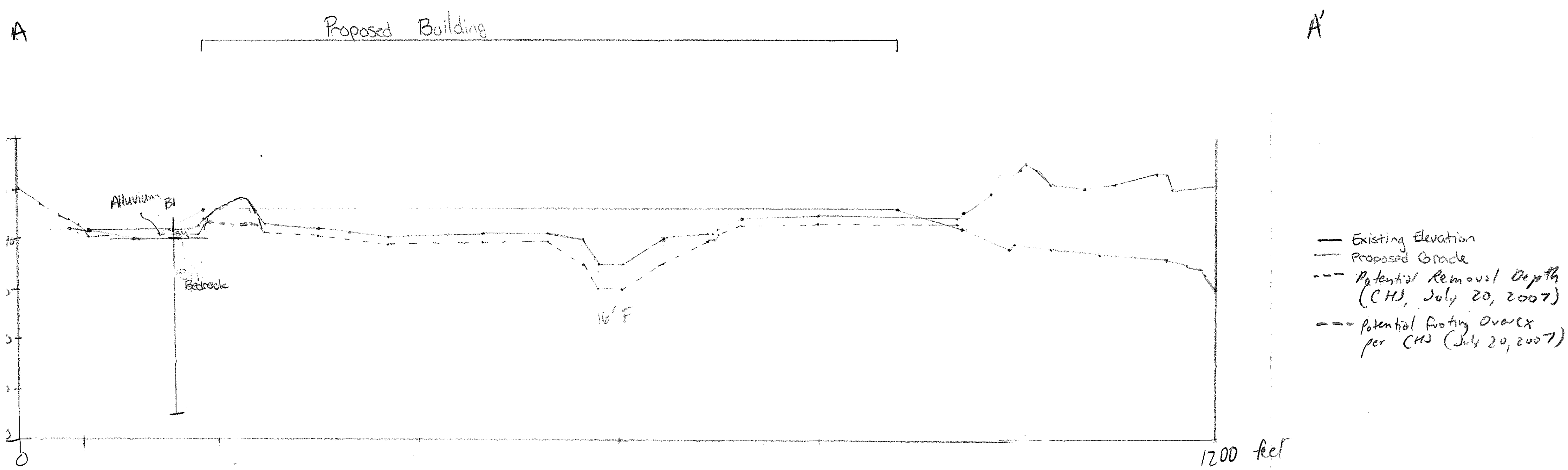
GEOLOGIC CROSS SECTION

N

SITE LOCATION MAP

FOR: THE MAGNON COMPANIES	SYCAMORE CANYON V PROJECT RIVERSIDE, CALIFORNIA	ENCLOSURE 1
DATE: JULY 2008		JOB NUMBER 07489-3

A Job #07849-3 Client: Magnon 7/08 H: 1"=100' V: 1"=20' A'



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JOB NO.: 07489-3

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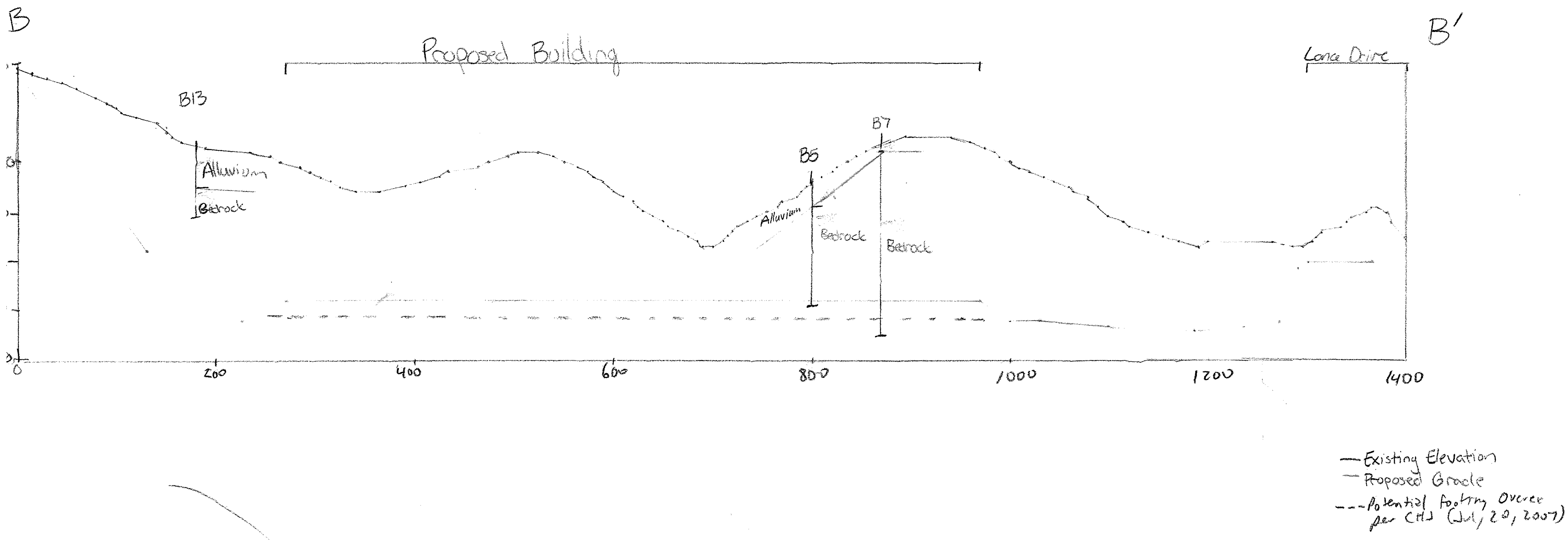

ENCLOSURE: 3
JOB NO.: 07489-3

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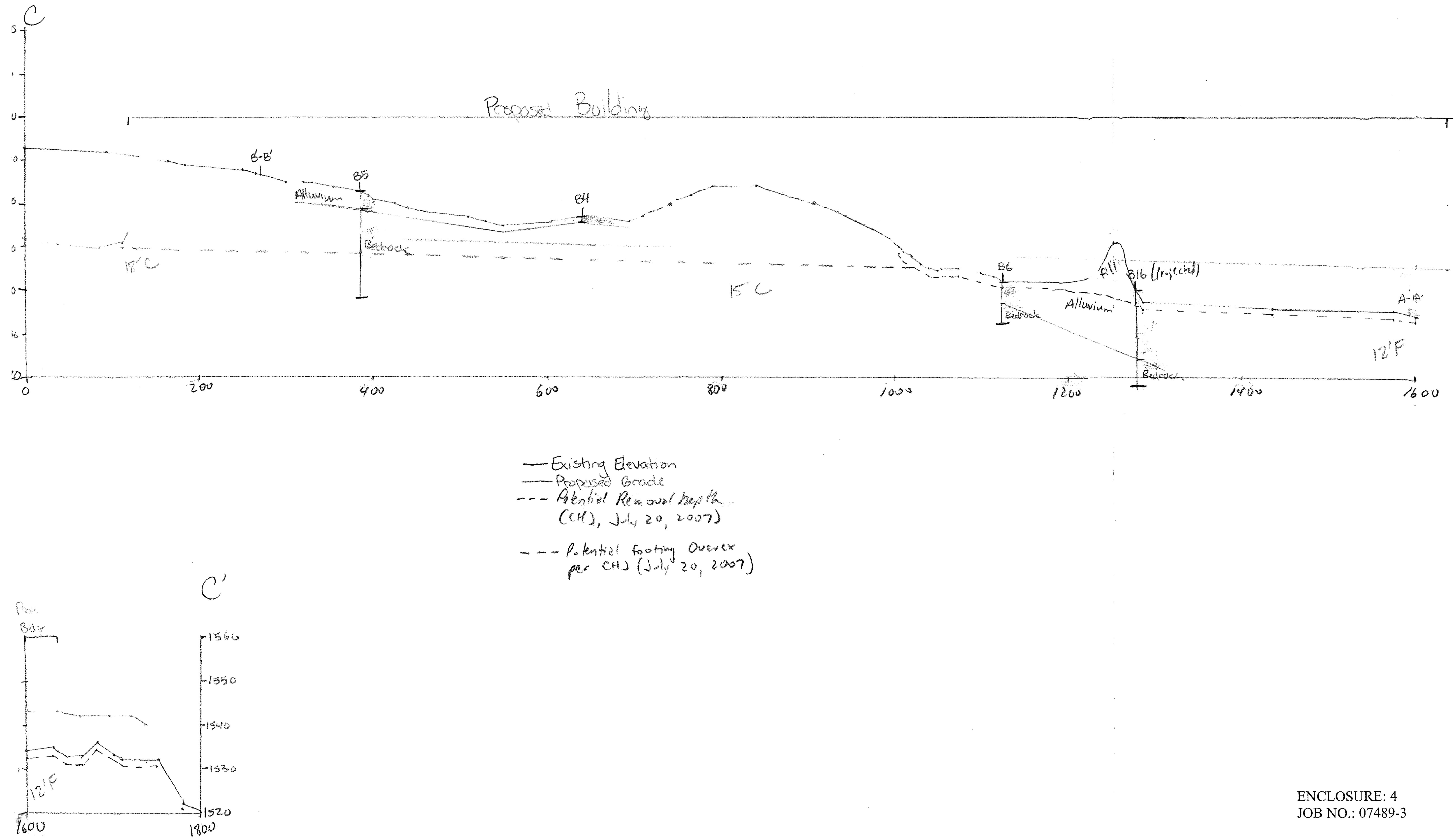

ENCLOSURE: 4
JOB NO.: 07489-3

GEOTECHNICAL INVESTIGATION
PROPOSED INDUSTRIAL DEVELOPMENT
LANCE AND SIERRA RIDGE DRIVES
RIVERSIDE, CALIFORNIA
PREPARED FOR
THE MAGNON COMPANIES
JOB NO. 07489-3

5" V, VI, VII (73-Acre)

C.H.J. Incorporated
1355 E. Cooley Drive, Colton, CA 92324 ◆ Phone (909) 824-7210 ◆ Fax (909) 824-7209
15345 Anacapa Road, Suite D, Victorville, CA 92392 ◆ Phone (760) 243-0506 ◆ Fax (760) 243-1225

July 20, 2007

The Magnon Companies
815 Marlborough Avenue, Suite 200
Riverside, California 92507
Attention: Mr. David Stapley

Job No. 07489-3

Dear Mr. Stapley:

Attached herewith is the Geotechnical Investigation report prepared for the proposed industrial development, to be located northwest of the intersection of Lance Drive and Sierra Crest Drive in the Sycamore Canyon area of Riverside, California.

This report was based upon a scope of services generally outlined in our proposal letter, dated June 14, 2007, and other written and verbal communications.

We appreciate this opportunity to provide geotechnical services for this project. If you have questions or comments concerning this report, please contact this firm at your convenience.

Respectfully submitted,
C.H.J., INCORPORATED

Ben Williams

Ben Williams, P.G. 7542
Senior Staff Geologist

BW:ndt

Distribution: The Magnon Companies (6)

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GEOTECHNICAL INVESTIGATION
PROPOSED INDUSTRIAL DEVELOPMENT
LANCE AND SIERRA RIDGE DRIVES
RIVERSIDE, CALIFORNIA
PREPARED FOR
THE MAGNON COMPANIES
JOB NO. 07489-3

INTRODUCTION

During July of 2007, a geotechnical investigation was performed by this firm for the proposed industrial development, to be located northwest of the intersection of Lance Drive and Sierra Ridge Drive in the Sycamore Canyon area of Riverside, California. The purpose of this investigation was to explore and evaluate the geotechnical conditions at the subject site and to provide appropriate geotechnical recommendations for design of the proposed structures and infrastructure.

To orient our investigation at the site, a master plan of the Sycamore Canyon Complex was provided for our use. To further delineate the subject site, aerial photographs were utilized. The approximate location of the site is shown on the attached Index Map (Enclosure "A-1").

The results of our investigation, together with our conclusions and recommendations, are presented in this report.

SCOPE OF SERVICES

The scope of services provided during this geotechnical investigation included the following:

- Review of published and unpublished literature and maps
- Review and analysis of stereoscopic aerial photographs flown from 1962 to 1995
- Placement of 12 seismic refraction lines within the site boundaries
- Analysis of seismic data in order to evaluate the rippability of the bedrock material
- Geologic field reconnaissance of the site and surrounding area
- Placement of 16 exploratory borings on the site
- Logging and sampling of exploratory borings as well as surficial soils for testing and evaluation

- Laboratory testing on selected samples
- Evaluation of the geotechnical engineering data to develop site-specific recommendations for site preparation, conventional static foundation design, and mitigation of potential geotechnical constraints.

PROJECT CONSIDERATIONS

It is our understanding that the subject site is to be developed with industrial/warehouse/distribution center type structures and related infrastructure. It is anticipated that the site will be developed with one- to two-story structures of concrete tilt-up construction, utilizing conventional spread foundations for support. Moderate foundation loading is anticipated for the structures.

The project grading plan was not available at the time of our investigation. The general topography indicates that the grading of this site will entail maximum cuts and fills on the order of 30 to 40 feet. The final grading plan should be reviewed by the geotechnical engineer.

SITE DESCRIPTION

The subject site is a rectangular-shaped parcel of approximately 75± acres located northwest of the intersection of Sierra Ridge Drive and Lance Drive in the Sycamore Canyon Development of the City of Riverside, California. The site is bounded by existing commercial developments to the east and south, a residential development to the north and the northwest, and vacant land to the west.

At the time of our investigation, the site was vacant with slightly rolling topography. Drainage was generally toward the center of the parcel to a shallow ravine and then to the south, with maximum slope inclinations on the order of 6 to 8 percent. Portions of the site were used for burrow purposes in the past, and evidence of minor cut/fill grading was visible in the southern portions of the site. A rock crusher operation existed in the southeast portion of the parcel.

Sparse vegetation consisting of annual weeds and grasses was growing on the site. No significant debris was observed. It is anticipated that areas of undocumented fill may exist associated with the crusher operation.

Review of our in-house stereoscopic aerial photographs dating back to 1962 show the site to be vacant. The site had a slightly rolling topography that was moderately incised by ephemeral drainages with a southern trend. The 1995 photographs show the site similar to its current condition.

No other surface features pertinent to this investigation were noted.

FIELD INVESTIGATION

A site reconnaissance was performed by representatives of this firm and our geophysical consultant on June 25, 2007. At that time, twelve locations were selected for placement of seismic refraction lines. The seismic refraction survey was conducted by Terra Geosciences on the following Saturday to limit the influence of background noise from rock crushing activity on-site and nearby construction. The results of the seismic refraction survey are included as Appendix "E".

The soil conditions underlying the subject site were explored by means of 16 exploratory borings drilled to a maximum depth of 37 feet below the existing ground surface with a truck-mounted CME 55 drill rig equipped for soil sampling. The approximate locations of our exploratory borings are indicated on the attached Plat (Enclosure "A-2").

Continuous logs of the subsurface conditions, as encountered within the exploratory borings, were recorded at the time of drilling by a staff geologist from this firm. Both a standard penetration test (SPT) sampler (2-inch outer diameter and 1 3/8-inch inner diameter) and a ring sampler (3 1/4-inch outer diameter and 2.42 inch inner diameter) were utilized in our investigation. The penetration resistance was recorded on the boring logs as the number of hammer blows used to advance the sampler in 6-inch increments (or less if noted). The samplers were driven with an automatic hammer that drops a 140-pound weight 30 inches for each blow. After the required seating, samplers are advanced up to 18 inches providing up to three sets of blowcounts at each sampling interval. The recorded blows are raw numbers without any corrections such as hammer type (automatic vs. manual cathead) or sampler size (ring sampler vs. SPT sampler). Relatively undisturbed as well as bulk samples of typical soil types obtained were returned to the laboratory in sealed containers for testing and evaluation.

Our exploratory boring logs, together with our in-place blowcounts per 6-inch increment, are presented in Appendix "B". The stratification lines presented on the boring logs represent approximate boundaries between soil types, which may include gradual transitions.

LABORATORY INVESTIGATION

Included in the laboratory testing program were field moisture content tests on all samples returned to the laboratory and field dry density tests on all relatively undisturbed samples. The results are included on the boring logs. Optimum moisture content - maximum dry density relationships were established for typical soil types. Direct shear testing was performed on selected remolded samples in order to provide shear strength parameters for bearing capacity and earth pressure evaluations. Sieve analyses and sand equivalent tests were performed on selected samples to aid in soil classification. Expansion index testing (UBC 18-2) was performed on a selected sample of clay bearing soil in order to evaulate the soil expansion characteristics. R-value testing was performed on a sample of probable pavement subgrade soil to assist in preliminary pavement structural section design. Selected samples of material were delivered to our corrosivity consultant for testing.

Summaries of the laboratory test results appear in Appendix "C".

SITE GEOLOGY AND SUBSURFACE SOIL CONDITIONS

The site is located on the Perris Block, a portion of the Peninsular Ranges Geomorphic Province. The Perris Block is a fault-bounded region of relative tectonic stability, a mass of relatively high land composed of crystalline bedrock of the southern California batholith that is thinly and discontinuously mantled by sedimentary material (Woodford and others, 1971). Several geomorphic surfaces are well developed on the Perris Block that represent former, local, erosional/depositional base levels. The site is located on the youngest of the Perris Block surfaces, known as the Paloma surface (Woodford and others, 1971). The Paloma surface is characterized by crystalline bedrock overlain by fine-grained alluvial deposits.

Regional geologic mapping conducted by Morton and Cox (2001) of the site and surrounding area, show the site as underlain by Val Verde Tonalite (granitic bedrock). Granitic bedrock was exposed in scattered areas throughout the site.

Data from our exploratory borings indicated that the subject site is generally blanketed by a layer of silty sand, underlain by granitic bedrock. In the areas explored, the silty sand (SM) is medium dense to dense and on the order of zero to 17 feet in thickness. Localized areas of clay bearing silty sand (SM) were encountered.

The upper soils were generally granular and are not anticipated to have a significant expansion potential. In localized areas, clay bearing silty sands were encountered. Expansion index testing (UBC 18-2) was performed on a selected sample of clay bearing soil. The results of our test, included in Appendix "C", indicate a "very low" expansion potential for these clay bearing soils.

The underlying bedrock is in a very dense state. Due to the grinding action of the augers the bedrock material was recovered as silty sand. Bedrock was encountered in all of the exploratory borings.

It appears that the bedrock is at or near the surface over a large portion of the site. In only four of the sixteen borings, was bedrock material encountered at a depth of 4 feet or greater. Within Exploratory Boring No. 16 the bedrock was encountered at a depth of 17 feet.

Fill to a depth of 11 feet was encountered in Exploratory Boring No. 16 associated with road construction across a stream channel. Fill was not encountered in the remaining borings utilized for this investigation. It is anticipated that undocumented fill, associated with previous site usage, may exist in localized areas of the site.

Refusal was encountered within all exploratory borings utilized for this investigation. The depth of refusal varied from 1 to 37 feet below the existing ground surface.

Groundwater was not encountered within any of our exploratory borings.

Our exploratory borings experienced only slight caving upon removal of the drilling augers.

The results of our soluble sulfate testing indicate a "negligible" exposure to sulfate attack according to the American Concrete Institute. The soils tested are classified as moderately corrosive to corrosive to ferrous metals. Further information on the corrosivity analysis is included in the Chemical/Corrosivity Tests section of this report and Appendix "C".

The locations of the exploratory borings are shown on the attached Plat (Enclosure "A-2").

A more detailed description of the subsurface soil conditions encountered within our exploratory borings is presented on the attached boring logs (Appendix "B").

FAULTING

The tectonics of the Southern California area are dominated by the interaction of the North American plate and the Pacific plate, which are apparently sliding past each other in a translational manner. Although some of the motion may be accommodated by rotation of crustal blocks such as the western Transverse Ranges (Dickinson, 1996), the San Andreas fault zone is thought to represent the major surface expression of the tectonic boundary and to be accommodating most of the translational motion between the Pacific plate and the North American plate. However, some of the plate motion is apparently also partitioned out to the other northwest-trending strike-slip faults that are thought to be related to the San Andreas system, such as the San Jacinto fault and the Elsinore fault. Local compressional or extensional strain resulting from the translational motion along this boundary is accommodated by left-lateral, reverse, and normal faults such as the San Jose fault, the Cucamonga fault zone, and the Crafton Hills fault zone (Matti and others, 1992; Morton and Matti, 1993).

The site does not lie within or adjacent to an Alquist-Priolo Earthquake Fault Zone designated by the State of California to include traces of suspected active faulting. No evidence for active faulting on the site was observed during the geologic field reconnaissance or on the aerial photographs reviewed.

The San Jacinto fault zone, a system of northwest-trending, right-lateral, strike-slip faults, is present approximately 6 miles northeast of the site. In the San Jacinto Valley, southeast of the site, the San Jacinto fault is expressed as two parallel main strands. The western strand, known as the Casa Loma fault, is well expressed in late Holocene alluvium in the Hemet area, but evidence of recent activity decreases to the north. North of the San Jacinto River, Rogers (1966) mapped the Casa Loma fault as a buried trace that links with the Reche Canyon fault as mapped by Morton (1978). That fault is approximately 11 miles north of the site. The eastern strand, or Claremont fault, is well expressed in the northern San Jacinto Valley at a closest distance of approximately 12 miles.

A maximum moment magnitude (**Mmax**) earthquake of **M** 6.9 is assigned to a rupture of the San Jacinto Valley segment of the San Jacinto fault (Petersen and others, 1996). More large historic earthquakes have occurred on the San Jacinto fault than any other fault in Southern California (Working Group on California Earthquake Probabilities, 1988). Based on the data of Matti and others (1992), this portion of the San Jacinto fault may be accommodating much of the motion between the Pacific plate and the North American plate in this area. Matti and others (1992) suggest this motion is transferred to the San

Andreas fault in the Cajon Pass region by "stepping over" to parallel fault strands which include the Glen Helen fault. The Working Group on California Earthquake Probabilities (1995) tentatively assigned a 43 percent (±17 percent) probability of a major earthquake on the San Jacinto Valley segment of the San Jacinto fault for the 30-year interval from 1994 to 2024.

The San Andreas fault zone is located along the southwest margin of the San Bernardino Mountains, approximately 16 miles northeast of the site. The toe of the mountain front in the San Bernardino area roughly demarcates the presently active trace of the San Andreas fault, which is characterized by youthful fault scarps, vegetational lineaments, springs, and offset drainages. The Working Group on California Earthquake Probabilities (1995) tentatively assigned a 28 percent (±13 percent) probability to a major earthquake occurring on the San Bernardino Mountains segment of the San Andreas fault between 1994 and 2004.

The Elsinore fault zone is present approximately 17 miles southwest of the site. The Elsinore fault zone is composed of multiple *en echelon* and diverging fault traces and splays into the Whittier and Chino faults to the north. Although a zone of overall right-lateral deformation consistent with the regional plate tectonics, traces of the Elsinore fault zone form the graben of the Elsinore and Temecula Valleys. Holocene surface rupture events have been documented for several principal strands of the Elsinore fault zone (Saul, 1978; Rockwell and others, 1986; Wills, 1988).

HISTORICAL EARTHQUAKES

A map of recorded earthquake epicenters is included as Enclosure "A-4" (Epi Software, 2000). This map includes the Cal Tech database for earthquakes with magnitudes of 4.0 or greater from 1977 through 2007.

The San Jacinto fault is the most seismically active fault in Southern California, although it has no record of producing great events comparable to those that occurred on the San Andreas fault during the Fort Tejon earthquake of 1857 and the San Francisco earthquake of 1906 (Working Group on California Earthquake Probabilities, 1988). Between 1899 and 1990, seven earthquakes of **M** 6 .0 or greater have occurred along the San Jacinto fault. Two of these earthquakes, an estimated **M** 6.7 in 1899 and an **M** 6.8 in 1918, took place in the San Jacinto Valley, southeast of the site. Two others, an estimated **M** 6.5 in 1899 and an **M** 6.2 in 1923, took place in the San Bernardino Valley, north of the site (Working Group on California Earthquake Probabilities, 1988).

No large earthquakes have occurred on the San Bernardino Mountains segment of the San Andreas fault within the regional historical time frame. Using dendrochronological evidence, Jacoby and others (1987) inferred that a great earthquake on December 8, 1812 ruptured the northern reaches of this segment. Recent trenching studies have revealed evidence of rupture on the San Andreas fault at Wrightwood occurred within this time frame (Fumal and others, 1993). Comparison of rupture events at the Wrightwood site and Pallett Creek, and analysis of reported intensities at the coastal missions, led Fumal and others (1993) to conclude that the December 8, 1812 event ruptured the San Bernardino Mountains segment of the San Andreas fault largely to the southeast of Wrightwood, possibly extending into the San Bernardino Valley. The average recurrence interval for large earthquakes along the southern San Andreas fault at six paleoseismic sites is 182 years (Stone and others, 2002). Surface rupture occurred on the Mojave segment of the San Andreas fault in the great 1857 Fort Tejon earthquake. The Coachella Valley segment of the San Andreas fault was responsible for the 1948 **M** 6.5 earthquake in the Desert Hot Springs area and for the 1986 **M** 5.6 earthquake in the North Palm Springs area.

The only large historical earthquake that can definitely be attributed to the Elsinore fault was an **M** 6.0 event in 1910 in the Temescal Valley area. This event caused damage to structures from Corona to Wildomar (Weber, 1977). Since 1932, four **M** 4.0+ earthquakes have occurred along the Elsinore fault zone in the Santiago Peak area (Weber, 1977).

SEISMIC ANALYSIS

The precise relationship between magnitude and recurrence interval of large earthquakes for a given fault is not known due to the relatively short time span of recorded seismic activity. As a result, a number of assumptions must be made to quantify the ground shaking hazard at a particular site. Seismic hazard evaluations can be conducted from both a probabilistic and a deterministic standpoint. The probabilistic method is prescribed for seismic design by current building codes and was utilized to estimate the seismic hazard to the site during this investigation.

PROBABILISTIC HAZARD ANALYSIS:

The probabilistic analysis of seismic hazard is a statistical analysis of seismicity of all known regional faults attenuated to a particular geographic location. The results of a probabilistic seismic hazard analysis are presented as the annual probability of exceedance of a given strong motion parameter for a particular exposure time (Johnson and others, 1992).

For this report, the seismic hazard analysis computer program EZFRISK, Version 7.22 (Risk Engineering, 2007) was used to analyze the location of the site under the criteria for rock sites. The estimated value for the peak ground acceleration was calculated as the average of the accelerations computed using the attenuation relations of Boore et al. (1997), Sadigh et al. (1997), and Abrahamson and Silva (1997), in relation to seismogenic faults within a 93-mile (150-km) radius of the site. The EZFRISK program considers seismicity from mappable seismogenic faults and background sources (those earthquakes not associated with a mapped fault source) and assumes that the occurrence rate of earthquakes on a fault is proportional to the estimated slip rate of that fault. Potential earthquake magnitudes are correlated to expected seismic sources, and the resultant maximum ground acceleration at the site is computed.

Based on the site-specific PSHA performed for the site, the estimated peak horizontal ground acceleration of 0.62g has a 10 percent probability of exceedance in 50 years (statistical return period of 475 years). This corresponds to the Design Basis Earthquake as defined in the 2001 California Building Code (CBC).

SEISMIC ZONE:

The site is included within Seismic Zone 4. Table 16-I of the 2001 (CBC) assigns a Seismic Zone Factor "Z" of 0.4 to Seismic Zone 4.

SOIL PROFILE CHARACTERIZATION:

Based on review of the boring logs, the anticipated grading, and the geologic setting, a soil profile type S_C, very dense soil and soft rock, is appropriate for the site.

NEAR-SOURCE EFFECTS:

The San Jacinto fault is classified as a Type "B" fault by the State of California. For the Type "B" classified San Jacinto fault, at a surface distance of approximately 6 miles (9 1/2 kilometers), the near-source acceleration factor N_A, as defined in the 2001 CBC, is 1.00, and the near-source velocity factor N_V is 1.02.

GROUNDWATER AND LIQUEFACTION

Liquefaction is a process in which strong ground shaking causes saturated soils to lose their strength and behave as a fluid (Matti and Carson, 1991). Ground failure associated with liquefaction can result in severe damage to structures. The geologic conditions for increased susceptibility to liquefaction are 1) shallow groundwater (generally less than 50 feet in depth), 2) presence of unconsolidated sandy alluvium, typically Holocene in age, and 3) strong ground shaking. All three of these conditions must be present for liquefaction to occur.

The site is underlain at shallow depths by dense to very dense granitic bedrock. Due to the presence of shallow bedrock and the absence of shallow groundwater, liquefaction is not considered to be a hazard at this site.

Groundwater was not encountered within the exploratory borings. There is a potential for perched, seasonal groundwater along the bedrock/alluvium interface due to the presence of relatively impermeable shallow bedrock overlain by more permeable alluvium.

FLOODING AND EROSION

No evidence of recent significant flooding of the site was observed during the geologic field reconnaissance or on the aerial photographs reviewed.

The upper soils encountered within the site consist of silty sands that are moderately susceptible to erosion by wind and water. Positive drainage should be provided and water should not be allowed to pond anywhere on the site. Water should not be allowed to flow over any graded or natural areas in such a way as to cause erosion.

A shallow stream channel traverses the site. Surface flow was not visible in the channel at the time of our investigation.

SLOPE STABILITY

Current site topography consists of rolling hills with approximately 60 feet of elevation difference across the site. Based on review of the proposed development, slopes up to 20 feet are likely. Geologic mapping conducted by Morton and Cox (2001) show regional jointing dipping moderately to the northeast. The bedrock outcrops at the site are highly weathered and did not exhibit preferred jointing orientations.

The term "landslide", as used in this report, refers to deep-seated slope failures at least 15 feet deep. Landslides are typically related to the underlying structure of the parent material. Surficial failures refer to shallow failures that affect the upper weathered horizon of the parent material. No evidence for deep-seated landsliding was observed during the field reconnaissance or on the aerial photographs reviewed.

The susceptibility of a geologic unit to landsliding is dependent upon various factors, primarily: 1) the presence and orientation of weak structures, such as fractures, faults and joints; 2) the height and steepness of the pertinent natural or cut slope; 3) the presence and quantity of groundwater; and 4) the occurrence of strong seismic shaking. No adversely-oriented structures were observed. Groundwater is not expected within any of the potential cut slopes.

Cut slopes up to 30 feet in height constructed entirely within the bedrock unit are considered to be stable against gross failure at inclinations no steeper than 2 horizontal to 1 vertical [2(h):1(v)].

Fill slopes constructed to a maximum of 30 feet and at inclinations no steeper than 2(h):1(v) are considered stable.

RIPPABILITY AND ROCK DISPOSAL

A seismic refraction survey was conducted by Terra Geosciences to analyze the rippability of the underlying bedrock. The results of the survey are summarized below and discussed in more detail in the attached report (Appendix "E"). Additionally, Exploratory Boring Nos. 4, 5, 10, and 11 were placed along the seismic lines to compare refusal depths with measured velocities of the seismic refraction survey. Based on comparison of the boring logs and the tomographic velocity models, refusal generally occurred at velocities ranging from 4,000 to 5,000 feet per second.

Measured seismic velocities at the site were varied. In general, rock with higher velocities was encountered at shallower depths in the southern portion of the site. No grading plans were available during the preparation of this report. Based on the anticipated grading, specialized excavation methods (such as blasting) may be required.

The seismic velocity can be utilized to estimate the rippability of subsurface materials. A chart included in the Caterpillar Performance Handbook (1992) correlates seismic velocity of different rock types to rippability by D8L or D9N bulldozers utilizing single- or multi-shank rippers. This chart indicates granitic rock, such as that which underlies the site, is rippable to a velocity up to 6,800 feet per second (fps). Granitic rock of between 6,800 and 8,000 fps is considered to be marginally rippable, and velocities of greater than 8,000 fps are considered to be non-rippable. Our experiences with similar sites of this type indicate that the velocities in the Caterpillar Performance Handbook (1992) are approximately 1,500 fps too fast for reasonable production rates.

No published data are available to relate the seismic velocity of rock with its excavation potential ("trenchability") for backhoes. A large excavator, such as a Cat 235, is typically used for larger utility trenching projects. A smaller rubber-tired backhoe, such as a Case 580, is typically used for smaller projects, such as residential utility lines and foundation trenching. Based upon our experience with granitic bedrock, a Cat 235 excavator can excavate granitic rock with velocities up to approximately 4,300 fps. A Case 580 backhoe can excavate granitic rock with velocities up to approximately 3,300 fps. Bedrock that is not trenchable with a backhoe will generally require blasting, jackhammering, or use of a chain trencher for excavation.

ROCK DISPOSAL:

Grading of the site may result in the generation of rock with dimensions greater than 12 inches. No rock greater than 12 inches in dimension may be buried or placed in fills. within 10 feet of final grade If it is necessary to dispose of such material on-site, the boulders should not be placed in site fills except in areas designated as "rock disposal areas". Such "rock disposal areas" should be non-buildable areas or areas below the depth of 10 feet.

In order to properly compact soils in "rock disposal areas", the rock should be placed in windrows, in such a manner as to avoid nesting and to allow access for compaction equipment. The oversized material should be placed in lifts, alternating with lifts of soil without oversized material. Continuous observation by the geotechnical engineer should be provided to confirm that adequate compaction is achieved in such areas. A typical rock disposal detailed is included in Appendix "D".

Again, these recommendations are preliminary. Additional recommendations can be provided once the proposed grading is known. In any case, it is crucial that the geotechnical engineer be present to observe these operations. Further recommendations may be made in the field depending on the actual conditions encountered.

CONCLUSIONS

On the basis of our field and laboratory investigations, it is the opinion of this firm that the proposed construction is feasible from a geotechnical engineering standpoint, provided the recommendations contained in this report are implemented during grading and construction.

Based upon our field investigation and test data, it is our opinion that the undocumented fill, native soils and granitic bedrock will not, in their present condition, provide uniform or adequate support for the proposed structures and infrastructures. Our blowcount and density testing indicated variable in-situ conditions of these materials ranging from loose to very dense states. The soil profile at the site extends to depths of up to 17 feet, at which depth the soil is underlain by granitic bedrock. In other areas of the site, bedrock is exposed at the surface. These conditions may cause unacceptable differential and/or overall settlement upon application of the anticipated foundation loads. Site clearing can be expected to further aggravate the settlement-prone conditions.

Based on the anticipated grading, non-rippable bedrock may be encountered. To obtain finish grade elevations and/or the recommended criteria described below, specialized excavation methods such as fill mat blasting may be necessary.

Due to the elevation difference across the site and the varying depth of bedrock, it does not appear feasible for the foundations of the proposed structure to bear entirely in undisturbed bedrock. The foundations of the structures should not be permitted to span from soil to bedrock. To provide adequate support for the proposed structures, it is our recommendation that they be provided with a compacted fill mat beneath all footings and slabs. A compacted fill mat will provide a dense, uniform, high-strength soil layer to distribute the foundation loads over the underlying soils. Conventional spread foundations, either individual spread footings and/or continuous wall footings, may be utilized in conjunction with a compacted fill mat.

The construction of this compacted fill mat should include, at a minimum, the mandatory removal and replacement of the upper 18 inches of existing soils. Additional removals of both the soil and bedrock may be necessary to provide the recommended fill mat.

The bottom of this excavation should be observed by the engineering geologist prior to scarification to verify the complete removal of any undocumented fill material or disturbed native soils. This required removal, and subsequent scarification and recompaction, will facilitate the densification of the upper soils. It will also assist in the identification of undocumented fill soils.

Severe seismic shaking of the site can be expected during the lifetime of the proposed structures.

Groundwater was not encountered within any of our exploratory borings.

Native soils encountered below a depth of approximately 2 feet were generally in medium dense to dense state and are underlain by dense to very dense granitic bedrock. The hydroconsolidation potential at the site is considered to be very low.

Undocumented fill was encountered in Exploratory Boring No. 16 to a depth of approximately 11 feet below the ground surface. Below the fill, native soils were encountered to a depth of 17 feet bgs.

All of our exploratory borings experienced slight caving upon removal of the drilling augers.

Due to the shallow bedrock and the depth to groundwater beneath the site, liquefaction and other shallow groundwater hazards are not anticipated.

No evidence of recent significant flooding of the site was observed during the geologic field reconnaissance or on the aerial photographs reviewed.

The on-site soils are generally granular and are considered to be non-critically expansive. Expansion testing (UBC 18-2) performed on a sample of clay bearing soils indicated a "very low" expansion potential.

RECOMMENDATIONS

SEISMIC DESIGN CONSIDERATIONS:

Moderate seismic shaking of the site can be expected during the lifetime of the proposed structures. Therefore, the proposed structures should be designed accordingly.

A soil profile type S_C, very dense soil and soft rock, is appropriate for the site.

The site is subject to near-source acceleration and velocity factors (N_A and N_V) of 1.00 and 1.02, respectively, as defined in the 2001 (CBC).

CUT SLOPE CONSTRUCTION:

Cut slopes up to 30 feet in height should be constructed no steeper than 2(h): 1(v). Cut slopes higher than 30 feet, if proposed, should be evaluated by the engineering geologist and geotechnical engineer prior to and during construction.

FILL SLOPE CONSTRUCTION:

Fill slopes should be constructed no steeper than 2(h):1(v) up to approximately 30 feet in height. Fill slopes should be overfilled during construction and then cut back to expose fully-compacted soil. A suitable alternative would be to compact the slopes during construction and then roll the final slopes to provide dense, erosion-resistant surfaces.

Where fills are to be placed against existing slopes steeper than 5(h):1(v), the existing slopes should be benched into competent native materials to provide a series of level benches to seat the fill and to remove the compressive and permeable top soils. The benches should be a minimum of 8 feet in width, constructed at approximately 4-foot vertical intervals. In addition, a shear key should be constructed across the toe of the slope. The shear key should be a minimum of 15 feet wide and should penetrate a minimum of 2 feet beneath the toe of the slope into firm competent soils. A Typical Benching Detail is included in Appendix "D".

SITE DRAINAGE:

Site drainage should be designed to accommodate local runoff. Increasing pavement surface with local development should be considered in the design of drainage systems.

GENERAL SITE GRADING:

It is imperative that no clearing and/or grading operations be performed without the presence of a representative of the geotechnical engineer. An on-site pre-job meeting with the developer, the contractor, and the geotechnical engineer should occur prior to all grading-related operations. Operations undertaken at the site without the geotechnical engineer present may result in exclusions of affected areas from the final compaction report for the project.

Grading of the subject site should be performed, at a minimum, in accordance with these recommendations and with applicable portions of the CBC. The following recommendations are presented for your assistance in establishing proper grading criteria.

<u>INITIAL SITE PREPARATION</u>:

All areas to be graded should be stripped of significant vegetation, and other deleterious materials. These materials should be removed from the site for disposal. Any existing utility lines and/or other underground structures should be traced, removed, and rerouted from the grading areas.

All fill encountered during this investigation and any existing undocumented fill or loose disturbed soils encountered during construction should be completely removed, cleaned of significant deleterious materials, and may be reused as compacted fill. Deleterious materials encountered at this time should be removed and replaced with compacted fill.

To assist in undocumented fill identification and removal, it is our opinion that a minimum depth of 18 inches of existing soil below the existing ground surface within the building areas and other areas to be graded should be completely removed and cleaned of significant deleterious materials.

The removed soils may be reused as compacted fill. Deeper fills and/or deeper disturbed native soils requiring complete removal are expected to exist and should be anticipated. The bottom of this excavation should be observed by the engineering geologist to verify the complete removal of undocumented fill material and disturbed native soils. Following approval, the bottom soils should be scarified to a depth of approximately 12 inches, brought to between optimum moisture content and 2 percent above, and recompacted to at least 95 percent relative compaction (ASTM D 1557) prior to refilling the excavation to grade as properly compacted fill.

Cavities created by removal of subsurface obstructions such as utility lines should be thoroughly cleaned of loose soil, organic matter, and other deleterious materials, shaped to provide access for construction equipment, and backfilled as recommended for site fill. A lean sand cement slurry may be considered to fill void areas difficult to compact.

PREPARATION OF FILL AREAS:

Prior to placing fill and after the mandatory subexcavation operation, observation, and approval by the engineering geologist, the surface of all soil areas to receive fill should be scarified to a depth of approximately 12 inches. The scarified soils should be brought to between optimum moisture content and 2 percent above and recompacted to a relative compaction of at least 95 percent in accordance with ASTM D 1557.

PREPARATION OF FOOTING AREAS:

All footings should either rest upon at least 18 inches of properly compacted fill material. Foundations should not be permitted to span from soil to bedrock.

In areas where the required thickness of compacted fill is not accomplished by the mandatory subexcavation operation, the footing areas should be subexcavated to a depth of at least 18 inches below the proposed footing base grade. The subexcavation should horizontally extend beyond the footing lines a minimum distance of 5 feet at the base of the excavation. The soils in the bottom of this excavation should then be scarified to a depth of at least 12 inches, brought to between optimum moisture content and 2 percent above, and recompacted to at least 95 percent relative compaction in accordance with ASTM D 1557 prior to refilling the excavation to grade as properly compacted fill.

ROCK DISPOSAL:

Current building codes require that rocks or other irreducable material greater than 12 inches in maximum dimension not be placed in fill within 10 feet of final grade. Boulders greater than 12 inches in dimension should not be placed in site fills except in areas designated as "rock disposal areas". Such "rock disposal areas" should be non-buildable areas below the depth of 10 feet.

In order to properly compact soils in "rock disposal areas", the rock should be placed in windrows, in such a manner as to avoid nesting and to allow access for compaction equipment. The oversized material should be placed in lifts, alternating with lifts of soil without oversized material. Continuous observation by the geotechnical engineer should be provided to confirm that adequate compaction is achieved in such areas. A typical rock disposal detailed is included in Appendix "D".

COMPACTED FILLS:

The on-site soils should provide adequate quality fill material provided they are free from roots, other organic matter, and deleterious materials. Unless approved by the geotechnical engineer, rock or similar irreducible material with a maximum dimension greater than 12 inches should not be buried or placed in fills.

Although not anticipated, import fill should be inorganic, non-expansive granular soil free from rocks or lumps greater than 6 inches in maximum dimension. Sources for import fill should be observed and approved by the geotechnical engineer prior to their use.

Fill should be spread in near-horizontal layers, approximately 8 inches in thickness. Thicker lifts may be approved by the geotechnical engineer if testing indicates that the grading procedures are adequate to achieve the required compaction. Each lift should be spread evenly, thoroughly mixed during spreading to attain uniformity of the material and moisture in each layer, brought to between optimum moisture content and 2 percent above, and compacted to a minimum relative compaction of 95 percent in accordance with ASTM D 1557.

Based upon the relative compaction of the soils tested during this investigation, the relative compaction anticipated for compacted fill soils, we estimate a compaction shrinkage of approximately 5 to 10 percent. Therefore, 1.05 to 1.10 cubic yards of in-place soil material would be necessary to yield 1 cubic yard of properly compacted fill material. In addition, we would anticipate subsidence of approximately 0.1 foot. These values are exclusive of losses due to stripping, or the removal of other subsurface obstructions, if encountered, and may vary due to differing conditions within the project boundaries and the limitations of this investigation.

Shrinkage is not anticipated for the granitic bedrock material when utilized as fill. The bedrock, when excavated and used as for fill material, is anticipated to have a bulking effect on the order of 5 percent.

The values presented for shrinkage and subsidence are estimates only. Final grades should be adjusted, and/or contingency plans to import or export material should be made to accommodate possible variations in actual quantities during site grading.

SUBDRAINAGE:

Since a potential for shallow, perched groundwater exists on the site, the need for subdrains should be evaluated by the geotechnical engineer during grading.

EXPANSIVE SOILS:

Since most of the upper materials encountered during this investigation were generally granular and considered to be non-critical expansive, and clay bearing soils tested for expansion (UBC 18.2) indicated a "very low" potential, specialized construction procedures to specifically resist expansive soil forces are not anticipated at this time. Additional evaluation of soils for expansion potential should be conducted by the geotechnical engineer during the construction operation.

FOUNDATION DESIGN:

If the site is prepared as recommended, the proposed structures may be safely founded on conventional spread foundations, either individual spread footings and/or continuous wall footings, bearing on a minimum of 18 inches of compacted fill or embedded into competent bedrock. Footings should be a minimum of 12 inches wide and should be established at a minimum depth of 12 inches below lowest adjacent final subgrade level. For the minimum width and depth, footings may be designed for a maximum safe soil bearing pressure of 2,300 pounds per square foot for dead plus live loads. This allowable bearing pressure may be increased by 200 psf for each additional foot of width and by 500 psf for each additional foot of depth to a maximum safe soil bearing pressure of 3,500 psf for dead plus live loads. These bearing values may be increased by one-third for wind or seismic loading.

For footings thus designed and constructed, we would anticipate a maximum settlement of less than 1 inch. Differential settlement between similarly loaded adjacent footings is expected to be approximately half the total settlement.

LATERAL LOADING:

Resistance to lateral loads will be provided by passive earth pressure and base friction. For footings bearing against compacted fill or approved native soils, passive earth pressure may be considered to be developed at a rate of 375 psf per foot of depth. Base friction may be computed at 0.35 times the normal load. Base friction and passive earth pressure may be combined without reduction.

For preliminary retaining wall or shoring design purposes, a lateral active earth pressure developed at a rate of 35 psf per foot of depth should be utilized for unrestrained conditions. For restrained conditions, an at-rest earth pressure of 60 psf per foot of depth should be utilized. Surcharge can be treated as additional height of backfill. Assume one additional foot for every 125 psf of areal surcharge. These values should be verified prior to construction when the backfill materials and conditions have been determined and are applicable only to level properly drained backfill with no additional surcharge loadings. If inclined backfills are proposed, this firm should be contacted to develop appropriate active earth pressure parameters.

Foundation concrete should be placed in neat excavations with vertical sides, or the concrete should be formed and the excavations properly backfilled as recommended for site fill.

SLABS-ON-GRADE:

To provide adequate support, concrete slabs-on-grade should bear on a minimum of 18 inches of compacted soil. The soil should be compacted to 90 percent relative compaction. The final pad surfaces should be rolled to provide smooth, dense surfaces upon which to place the aggregate or gravel.

Slabs to receive moisture-sensitive coverings should be provided with a moisture vapor retarder. We recommend that a vapor retarder be designed and constructed according to the American Concrete Institute (ACI) 302.1R, Guide for Concrete Floor and Slab Construction, which addresses moisture vapor retarder construction. At a minimum, the vapor retarder should comply with ASTM E 1745 and have a nominal thickness of at least 10 mils. The vapor retarder should be properly sealed per the manufacturers recommendations and protected from punctures and other damage. One inch of sand under the vapor retarder may assist in reducing punctures.

If heavy slab loading such as materials storage and/or forklift traffic is anticipated, building slabs should be designed by a Registered Civil Engineer competent in concrete design.

POTENTIAL EROSION:

The potential for erosion should be mitigated by proper drainage design. Water should not be allowed to flow over graded areas or natural areas so as to cause erosion. Graded areas should be protected from erosion by wind or water.

PRELIMINARY PAVEMENT DESIGN:

As a part of this investigation, samples of probable pavement subgrade soils were returned to the laboratory for analysis. An R-value test was performed on a representative sample. Based on the results of the testing, we are providing preliminary recommendations for asphalt concrete (AC) and Portland cement concrete (PCC) pavement structural section design.

PRELIMINARY ASPHALT PAVEMENT DESIGN:

Street	T.I.	Recommended Street Section
Parking Areas	5.0	0.25' AC/0.45' AB
Truck Parking and Drive Traffic Areas	8.0	0.40' AC/0.85' AB

The above structural sections are predicated upon proper compaction of the utility trench backfills and the subgrade soils, with the upper 6 inches of subgrade soils and all aggregate base material brought to a relative compaction of at least 95 percent (ASTM D 1557) prior to paving. The aggregate base should meet Caltrans requirements for Class 2 base.

PRELIMINARY CONCRETE PAVEMENT DESIGN:

Based upon an R-value of 31, which correlates to a modulus of subgrade reaction of approximately 140 pounds per square inch per inch (k), we recommend the following PCC pavement designs. This design is based upon the American Concrete Institute (ACI) Guide for Design and Construction of Concrete Parking Lots (ACI 330R-01).

Design Area	Recommended Section
Car Parking and Access Lanes ADTT = 0 (Category A)	4.0" PCC/Compacted Soil
Truck Parking Areas Multiple Units ADTT = 100 (Category C)	7.0" PCC/Compacted Soil

The above recommended concrete sections are based on a design life of 20 years, with integral curbs or thickened edges. In addition, the above structural sections are predicated upon proper compaction of the utility trench backfills and the subgrade soils, with the upper 12 inches of subgrade soils brought to a uniform relative compaction of 95 percent (ASTM 1557).

Thickened edges or integral curbs should be utilized to strengthen the edges of the slabs to enhance load-carrying capacity. Thickened edges should be thickened 2 inches at the outside edge and tapered to 36 inches back from the edge. Typical details are given in the referenced ACI publication. Thickened edges should only be necessary in areas where vehicles are allowed to travel from the new pavement to an existing slab, or from the new slab to soil, or asphalt concrete pavement.

The concrete sections may be placed directly over a compacted subgrade prepared as described above. The concrete to be utilized for the concrete pavement should have a minimum modulus of rupture of 550 pounds per square inch. This approximates a 28-day compressive strength of approximately 3,650 pounds per square inch. However, the design strength should be based upon the flexural strength (M_R) and not the compressive strength. Contraction joints should be sawcut in the pavement at maximum spacing of 30 times the thickness of the slab, up to a maximum of 15 feet. Sawcutting in the pavement should be performed within 12 hours of concrete placement, or preferably sooner. Sawcut depths should be equal to approximately one-quarter of the slab thickness for conventional saws or 1 inch when early-entry saws are utilized on slabs 9 inches thick or less. The use of plastic strips for formation of jointing is not recommended. The use of expansion joints is not recommended, except where the pavement will adjoin structures. Construction joints should be placed such that adjacent sections butt directly against each other and are keyed into each other or the joints are properly doweled with smooth dowels. It should be noted that distributed steel reinforcement (welded wire fabric) is not necessary, nor will any decrease in section thickness result from its inclusion.

The above pavement designs are based upon the results of preliminary sampling and testing, and should be verified by additional sampling and testing during construction when the actual pavement subgrade soils are exposed. In addition, if different ADTTs, traffic categories or concrete strengths are to be utilized, this firm should be contacted in order to provide revised recommendations.

C.H.J., Incorporated does not practice traffic engineering. The ADTTs used to develop the recommended PCC pavement sections are typical for projects of this type. We recommend that the ADTTs used be reviewed by the project civil engineer or traffic engineer to verify that they are appropriate for this project.

CHEMICAL/CORROSIVITY TESTS:

A selected sample of material was delivered to our corrosion consultant, Anaheim Test Laboratory. Laboratory testing consisted of pH, resistivity, and major soluble salts commonly found in soils. The results of the laboratory tests appear in Appendix "C". These tests have been performed in order to provide a preliminary screening of the site for potentially corrosive soils.

Values obtained indicate that the soils tested are considered potentially moderately corrosive to corrosive to ferrous metals.

Results of the soluble sulfate testing indicate a "negligible" anticipated exposure to sulfate attack, as indicated in Appendix "C". Based upon the criteria from Table 4.3.4. of the American Concrete Institute Manual of Concrete Practice (2000), no special measures, such as specific cement types, water-cement ratios, etc., will be needed for this "negligible" exposure to sulfate attack.

Soluble chloride content of soil was not at levels high enough to be of concern with respect to corrosion of reinforcing steel. The results should be considered in combination with the soluble chloride content of the hardened concrete in determining the effect of chloride on the corrosion of reinforcing steel.

C.H.J., Incorporated does not practice corrosion engineering. If further information concerning the corrosion characteristics, or interpretation of the results submitted herein, are required, then a competent corrosion engineer could be consulted.

CONSTRUCTION OBSERVATION:

All grading operations, including site clearing and stripping, should be observed by a representative of the geotechnical engineer. The presence of the geotechnical engineer's field representative will be for the purpose of providing observation and field testing, and will not include any supervising or directing of the actual work of the contractor, his employees, or agents. Neither the presence of the geotechnical engineer's field representative nor the observations and testing by the geotechnical engineer shall excuse the contractor in any way for defects discovered in his work. It is understood that the geotechnical engineer will not be responsible for job or site safety on this project, which will be the sole responsibility of the contractor.

LIMITATIONS

C.H.J., Incorporated has striven to perform our services within the limits prescribed by our client, and in a manner consistent with the usual thoroughness and competence of reputable geotechnical engineers and engineering geologists practicing under similar circumstances. No other representation, express or implied, and no warranty or guarantee is included or intended by virtue of the services performed or reports, opinion, documents, or otherwise supplied.

This report reflects the testing conducted on the site as the site existed during the investigation, which is the subject of this report. However, changes in the conditions of a property can occur with the passage of time, due to natural processes or the works of man on this or adjacent properties. Changes in applicable or appropriate standards may also occur whether as a result of legislation, application, or the broadening of knowledge. Therefore, this report is indicative of only those conditions tested at the time of the subject investigation, and the findings of this report may be invalidated fully or partially by changes outside of the control of C.H.J., Incorporated. This report is therefore subject to review and should not be relied upon after a period of one year.

The conclusions and recommendations in this report are based upon observations performed and data collected at separate locations, and interpolation between these locations, carried out for the project and the scope of services described. It is assumed and expected that the conditions between locations observed and/or sampled are similar to those encountered at the individual locations where observation and sampling were performed. However, conditions between these locations may vary significantly. Should conditions be encountered in the field, by the client or any firm performing services for the client or the client's assign, that appears different from those described herein, this firm should be contacted immediately in order that we might evaluate their effect.

If this report or portions thereof are provided to contractors or included in specifications, it should be understood by all parties that they are provided for information only and should be used as such.

The report and its contents resulting from this investigation are not intended or represented to be suitable for reuse on extensions or modifications of the project, or for use on any other project.

CLOSURE

We appreciate this opportunity to be of service and trust this report provides the information desired at this time. Should questions arise, please do not hesitate to contact this office at your convenience.

Respectfully submitted,
C.H.J., INCORPORATED

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