APPENDIX B

Geotechnical Investigation Report-2007
TRC Solutions

Geotechnical Investigation

Campus at North First Street

Phase 1

San Jose, California

Report No. 1738-1B/153164.0 has been prepared for:

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GEOTECHNICAL INVESTIGATION

CAMPUS AT NORTH FIRST STREET

PHASE 1

SAN JOSE, CALIFORNIA

1.0 INTRODUCTION

In this report we present the results of our geotechnical investigation for the Campus at North First Street, Phase 1 to be located in San Jose, California. The location of the site is shown on the Vicinity Map, Figure 1. The purpose of our investigation was to evaluate the subsurface conditions at the site and to provide geotechnical recommendations for design of the proposed development. For our use we received an Overall Site Plan, Sheet A0.02 and a Phase 1 Site Plan, Sheet A1.00 titled "Campus at North First, Planned Development Permit," dated August 10, 2007, prepared by Arquitectonica. We also received Sheets 1 to 3 titled "Stockpile Grading Permit," dated March 29 and September 17, 2002, and a Topographic Exhibit titled "The Campus at North First Street," dated July 27, 2007, prepared by HMH, Incorporated.

1.1 Project Description

Tishman Speyer is considering developing the site with 2,820,000 square feet of buildings containing office, retail, and community development. We understand the new Campus at North First Street will be developed in four phases and Tishman Speyer intends to begin construction of the first phase in the second quarter of 2008. This report addresses the design for Phase 1 only.

Phase 1 will include only two of the eight office buildings and half of one parking structure in the northeast portion of the site. The two at-grade office buildings will be thirteen and twelve floors in height with building footprints of approximately 40,000 square feet each. The at-grade parking structure will be six stories. Phase 1 will also include a 15,000 square feet Fitness Center/Community Center. The first floors of all the buildings except the parking structure will be irregularly shaped as shown on the Site Plan, Figure 2.

1.2 Scope of Services

Our scope of services was presented in detail in our agreements with you dated September 28 and November 28, 2007. To accomplish this work, we provided the following services:

- Exploration of subsurface conditions by drilling eight hollow-stem auger borings and three rotary wash borings and retrieving soil samples for observation and laboratory testing. Seven Cone Penetration Tests (CPTs) were also advanced.

- Evaluation of the physical and engineering properties of the subsurface soils by visually classifying the samples and performing various laboratory tests on selected samples. Correlation of CPT interpretations with visual classification and laboratory testing on samples collected from our borings.

- Engineering analysis to evaluate site earthwork, building foundations, slabs-on-grade, retaining walls and pavements.

- Preparation of this report to summarize our findings and to present our conclusions and recommendations.

Environmental services were not included as part of this study.

2.0 SITE CONDITIONS

2.1 Exploration Program

We previously performed a geotechnical investigation at this site which included nine hollow-stem auger borings to a maximum depth of 35½ feet and 12 Cone Penetration Tests (CPTs) to a maximum depth of 45 feet. Our recent field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem auger drilling, rotary-wash drilling, and Cone Penetration Test (CPT) equipment. Subsurface exploration was performed on October 8 to 12, 2007, using conventional, truck-mounted hollow-stem drilling equipment and on October 18 and 19, 2007 using Cone Penetration (CPT) equipment to investigate, sample, and log subsurface soils. We performed three additional rotary wash borings next to previously performed CPT-1, CPT-3, and CPT-4 on November 27, 2007 to collect samples in the liquefiable sand layers. Eleven exploratory borings and seven CPTs were advanced to depths ranging from 30 to 130 feet. Our borings and CPTs were permitted and backfilled in accordance with Santa Clara Valley Water District and City of San Jose guidelines. The approximate locations of the borings and CPTs are shown on the Site Plan, Figure 2. Logs of our borings and CPTs, and details regarding our field investigation are included in Appendix A; our laboratory tests are discussed in Appendix B.

2.2 Surface

The 41-acre site is located southwest of the intersection of North First Street and Component Drive in San Jose, California. The project site is surrounded by North First Street to the north, Orchard Parkway to the south, commercial development to the east, including BEA Systems and Ebay, and a vacant lot is to the west of the site across Orchard Parkway. Component Drive runs down the middle of the site. The site is currently undeveloped and relatively level except for the five large stockpiles of soil near Orchard Parkway. The Guadalupe River is approximately 1/2 mile west of the site.

On Wednesday December 13, 2007 we performed a visual evaluation of the existing stockpiles at the site as shown on sheet 3 titled, “Stockpile Grading Permit,” dated September 17, 2002, prepared by HMH, Incorporated. Two stockpiles, 7,000 and 9,300 cubic yards (CY), are located south of Component Drive near Orchard Parkway. Three stockpiles, 3,200, 5,100, and 5,900 CY, are located north of Component Drive near Orchard Parkway. Based on our visual evaluation, the 9,300 CY stockpile south of Component Drive and 3,200 CY stockpile north of Component Drive may be used as structural fill and/or non-expansive fill (NEF) subject to additional laboratory testing.

2.3 Subsurface

Our explorations generally encountered medium stiff to hard lean clay with interbedded medium dense sand layers to 130 feet, the maximum depth explored. The sand layers encountered were up to 27½-feet-thick. Based on washed sieve analyses, the sands encountered contain between 8 and 49 percent fines.

Plasticity Index (PI) tests were performed on surface samples from Borings EB-6 and EB-8 at a depth of 2 and 4 feet, respectively. The tests resulted in PI's of 44 and 46, indicating very high plasticity and expansion potential of the near surface soils.

Plasticity Index (PI) tests were performed on deeper samples from Borings RW-1, RW-2, and RW-3 at depths of 26½, 39½, and 26½ feet, respectively. The tests resulted in PI's of 5, 7, and 8, indicating low plasticity.

2.4 Ground Water

Free ground water was encountered during subsurface exploration in all of the exploratory borings at depths ranging from 8½ to 23 feet; pore pressure measurements taken in our CPTs indicated depth to ground water ranging from 1½ to 10 feet. Please note the ground water depth measurements were taken at the time of drilling for all projects reviewed as well as the current borings and may not reflect a stabilized level. All borings and CPTs were backfilled immediately after drilling. Ground water maps published in the California Geologic Survey (CGS) seismic report of the Milpitas Quadrangle indicate that historically high ground water depths have been recorded at about 5 feet in the vicinity. We judge a design ground water depth of 5 feet below the ground surface to be appropriate for geotechnical design purposes, including liquefaction evaluation. Fluctuations in the level of the ground water may occur due to variations in rainfall, underground drainage patterns, and other factors not evident at the time measurements were made.

3.0 GEOLOGIC HAZARDS

A brief qualitative evaluation of geologic hazards was made during this investigation. Our comments concerning these hazards are presented below.

3.1 Fault Rupture Hazard

The San Francisco Bay Area is one of the most seismically active regions in the United States. The significant earthquakes that occur in the Bay Area are generally associated with crustal movement along well-defined, active fault zones of the San Andreas Fault system, which regionally trend in a northwesterly direction. Table 1 lists the active faults within 25 kilometers of the site:

Table 1. Approximate Distance to Seismic Sources

Fault	Distance (miles)	Closest Distance (kilometers)
Hayward (Southeast Extension)	4.7	7.6
Hayward (Total Length)	7.5	12.0
Calaveras	8.1	13.0
Monte Vista - Shannon	9.2	14.8
San Andreas (1906)	12.9	20.8

A Regional Fault Map illustrating known active faults relative to the site is presented in Figure 3. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone (known

formerly as a Special Studies Zone) or a Santa Clara County Fault Hazard Zone. As shown on Figure 3, no known surface expression of active faults is believed to cross the site. Fault rupture through the site, therefore, is not anticipated.

3.2 Estimated Ground Shaking

The seismic history of the region and studies of recurrence intervals of major faults, such as the San Andreas Fault, indicate that the site is likely to experience strong shaking from a significant earthquake during the design life of the planned development. In addition to estimating the probability of an earthquake, the anticipated level of ground shaking is important to know for the purposes of designing buildings and other improvements.

We performed a Probabilistic Seismic Hazard Analysis (PSHA) for this site to estimate the expected mean peak ground acceleration (PGA) using the computer software EZ-FRISK (7.23) developed by Risk Engineering, Incorporated, 2002 USGS California Fault and background seismicity models, and commonly accepted attenuation relationships of Abra.-Silva, Boore-Joyner-Fumal, Campbell, and Sadigh. The analysis was performed for known active faults located within a 100-kilometer radius of the site. Our analysis was performed to estimate seismic hazard shaking levels of 10 percent chance of exeedance in 50 years with 5 percent and 7½ percent damping.

Our probalistic seismic hazard analysis indicated that a rock site located at the project site has horizontal peak ground accelerations (PGA) of 0.52g and 0.62g, corresponding to seismic hazard levels of 10 percent chance of exeedance in 50 years and 10 percent change of exceedance in 100 years, respectively. These horizontal PGA values were selected as mean values generated using four commonly accepted attenuation relationships. Details regarding our site response analysis are presented below.

Magnitude 6 to 8 Mw events originating on the nearby Hayward and San Andreas Faults contribute significantly to the site seismic hazard.

3.3 Selected Ground Motion Records

We evaluated the estimated seismic response based on actual earthquake records, collected on rock, from seven earthquakes with magnitudes ranging from 6.1 to 7.6 Mw, at distance of less than 1 kilometer to about 17 kilometers from the fault rupture of the earthquakes. These records were recorded at sites that have been generally described as soft to hard rock, consistent with the bedrock conditions anticipated below the study site. These records were chosen as representative earthquakes to model the subject site, which is located about 12.0 kilometers from the Hayward (Total) Fault and about 20.8 kilometers from the San Andreas Fault. The San Andreas Fault is able to generate very large earthquakes, up to about magnitude 7.9(+/-) Mw. The magnitudes of earthquakes used in our evaluation were similar to the seismic events that contribute to significant seismic hazard for the site. The table below summarizes the earthquake records used for our analysis.

Table 2. Earthquake Records Used for Seismic Response Evaluation

Earthquake	Recording Station ID	Magnitude (Mw)	Distance to Rupture (km)	Record Source
Chi-Chi, Taiwan, 1999	CHY080	7.6	7.0	PEER Strong Motion Data Base
Duzce, Turkey, 1999	Duzce	7.1	8.2	PEER Strong Motion Data Base
Landers, California, 7.3	22170 Joshua Tree	7.3	11.6	PEER Strong Motion Data Base
Loma Prieta, California, 1989	58065 Saratoga-Aloha Ave	6.9	13.0	PEER Strong Motion Data Base
Northridge, California, 1994	24688 LA-UCLA Grounds	6.7	14.9	PEER Strong Motion Data Base
Parkfield, California, 1996	1013 Cholame #2	6.1	0.1	PEER Strong Motion Data Base
Kocaeli, Turkey, 1999	Sakarya	7.4	3.1	PEER Strong Motion Data Base

3.4 Response Analysis

The recorded earthquake accelerations were scaled to a horizontal peak ground acceleration of 0.52g, corresponding to the estimated bedrock accelerations for the site. We did not alter the frequency content of the earthquake records for each hazard level, since most of the variation of the PGA at these hazard levels is believed to come from uncertainties in peak ground acceleration attenuation relationships rather than uncertainty in controlling earthquake magnitude events. The scaled motions were input into a 1-D soil column to model the site soil conditions and estimate the surface response.

We evaluated the spectral acceleration response for each of the above earthquake records (for both x and y horizontal components when available) for an imaginary massless element with a damping ratio of 5 and 7½ percent for structural periods up to 5 seconds.

3.5 Results of Seismic Site Response Analysis

The response spectra presented in the tables below represent smoothed response spectra curves for the above referenced earthquake ground motion records scaled to the PGAs at 10 percent chance of exceedance in 50 years, as discussed above. The results are also presented graphically in Figures 4A and 4B.

Table 3. Horizontal Response Spectra

Period (seconds)	Horizontal Spectral Acceleration (g) 10% in 50 years	
	5 percent Damping	7½ percent Damping
0	0.57	0.57
0.1	0.8	0.75
0.3	0.98	0.89
0.5	1.08	0.96
0.7	1.18	1.01
0.9	1.21	1.04
1.15	1.19	1.02
1.4	1.17	0.98
1.65	1.12	0.92
1.9	1.03	0.85
2.5	0.85	0.72
3.0	0.75	0.62
3.5	0.65	0.55
4.0	0.55	0.5
4.5	0.5	0.45
5.0	0.45	0.4

3.6 Future Earthquake Probabilities

Although research on earthquake prediction has greatly increased in recent years, seismologists cannot predict when or where an earthquake will occur. The U.S. Geological Survey's Working Group on California Earthquake Probabilities (2003), referred to as WG02, determined there is a 62 percent chance of at least one magnitude 6.7 or greater earthquake striking the San Francisco Bay region between 2003 and 2032. This result is an important outcome of WG02's work, because any major earthquake can cause damage throughout the region.

The 1989 Loma Prieta earthquake demonstrated this potential by causing severe damage in Oakland and San Francisco, more than 50 miles from the fault rupture. Although earthquakes can cause damage at a considerable distance, shaking will be very intense near the fault rupture. Therefore, earthquakes located in urbanized areas of the region have the potential to cause much more damage than the 1989 Loma Prieta earthquake.

3.7 Liquefaction

The site is located within an area zoned by the State of California as having potential for seismically induced liquefaction hazards (CGS, 2004 – Milpitas Quadrangle) and in a Santa Clara County Geologic Hazard Zone (Santa Clara County, 2003) mapped liquefaction zone. During cyclic ground shaking, such as during earthquakes, cyclically induced stresses may cause increased pore water pressures within the soil matrix, resulting in liquefaction. Liquefied soil may lose shear strength that may lead to large shear deformations and/or flow failure under moderate to high shear stresses, such as beneath foundations or sloping ground (NCEER/NSF, 1998), and in many ways may behave more like a liquid than a solid. Liquefied soil can also settle (compact)

as pore pressures dissipate following an earthquake. Limited field data is available on this subject; however, in some cases, settlement on the order of 2 to 3 percent of the thickness of the liquefied zone has been measured.

Soils most susceptible to liquefaction are loose to moderately dense, saturated non-cohesive soils with poor drainage, such as sands and silts with interbedded or capping layers of relatively low permeability soil.

3.7.1 Analysis and Results

As noted in the subsurface description above, several sand and silt layers were encountered below the recommended design ground water depth of about 5 feet. These layers were evaluated to assess liquefaction potential and the effects liquefaction may have on the proposed structures.

Our liquefaction analyses followed the methods presented by the 1998 NCEER Workshops (Youd et al., 2001) in accordance with guidelines set forth in CDMG Special Publication 117 (CDMG, 1997). The NCEER methods for SPT and CPT analyses update simplified procedures presented by Seed and Idriss (1971).

In broad terms, these methods are used to calculate a factor of safety against liquefaction triggering by comparing the resistance of the soil to cyclic shaking to the seismic demand that can be caused during seismic events.

The resistance to cyclic shaking is quantified by the Cyclic Resistance Ratio (CRR), which is a function of soil density, layer depth, ground water depth, earthquake magnitude, and soil behavior. CRR calculations are based on SPT blow counts and CPT tip resistance. Our CPT tip pressures were corrected for overburden and fines content. The CPT method utilizes the soil behavior type index (I_C) and the exponential factor “n” applied to the Normalized Cone Resistance “Q” to evaluate how plastic the soil behaves.

The Cyclic Stress Ratio (CSR) is used to quantify the stresses that are anticipated to develop during cyclic shaking. The formula for CSR is shown below:

$$CSR = 0.65\ (a_{max}/g)(\sigma_{vo}/\sigma'_{vo})r_d$$

where a_{max} is the peak horizontal acceleration at the ground surface generated by an earthquake, g is the acceleration of gravity, σ_{vo} and σ'_{vo} are total and effective overburden stresses, respectively, and r_d is a stress reduction coefficient. We use a probabilistic pseudo-peak horizontal acceleration of 0.52g, corresponding to a 10 percent chance of exceedance in 50 years. Pseudo-peak ground accelerations have been normalized to a 7.5Mw seismic event, weighted to account for regional seismic activity and fault distances.

Soils that have significant amounts of plastic fines (greater than about 35 percent) or an I_C greater than 2.6, and corrected CPT tip resistances greater than 160 tons per square foot (tsf) are considered either too plastic or too dense to liquefy. Such soil layers have been screened out during our analysis and are not presented below.

The factor of safety (FS) against liquefaction can be expressed as the ratio of the cyclic resistance ratio (CRR) to the cyclic stress ratio (CSR). If the FS for a soil layer is less than 1.0, it is possible that the soil layer may liquefy during a moderate to large seismic event.

$$FS = CRR/CSR$$

A summary of our analysis for CPT data is presented in the tables below. We used blowcounts and laboratory testing from our rotary wash borings and shear wave velocities to further analyze the CPT data. An analysis was not performed on the SPT data collected in hollow-stem borings, since blow counts in hollow stem borings may be unreliable in sands below the ground water table.

Table 4. Results of Liquefaction Analyses – CPT Method

CPT Number	Depth to Top of Sand/Silt Layer (feet)	Layer Thickness (feet)	I_C	$*q_{C1N}$	Factor of Safety	Potential for Liquefaction	Estimated Total Settlement (in.)	Estimated Differential Settlement (in.)
CPT-1	26.2	0.5	2.3	45.9	0.2	Likely	0.3	0.2
	71.9	0.7	2.2	98.8	0.5	Likely	0.2	0.1
	82.3	2.3	103.2	103.2	0.6	Likely	0.6	0.3
	87.8	1.8	1.7	103.4	0.7	Likely	0.5	0.3
	96.8	0.5	1.9	117.7	0.8	Likely	0.1	<0.1
	98.4	0.5	1.7	120.7	0.9	Likely	<0.1	<0.1
						Total =	**1.7**	**0.9**
CPT-2	28.2	0.3	2.4	73.6	0.2	Likely	0.1	<0.1
	33.4	0.3	2.3	47.8	0.2	Likely	0.2	0.1
	45.6	0.3	2.2	71.1	0.2	Likely	0.1	<0.1
	68.2	3.3	1.9	111.0	0.7	Likely	0.8	0.4
	73.3	1.8	1.7	112.6	0.7	Likely	0.4	0.2
						Total =	**1.6**	**0.8**
CPT-3	25.4	0.3	2.4	41.8	0.2	Likely	0.2	0.1
	26.2	0.3	2.3	44.1	0.2	Likely	0.1	<0.1
	32.3	0.5	2.1	51.6	0.2	Likely	0.2	0.1
	61.7	0.7	2.2	59.1	0.3	Likely	0.3	0.2
	62.8	0.5	2.4	65.6	0.3	Likely	0.2	0.1
	64.1	0.7	2.2	58.5	0.3	Likely	0.3	0.2
	71.5	0.8	2.1	98.8	0.5	Likely	0.2	0.1
	91.5	0.7	1.9	101.5	0.6	Likely	0.1	<0.1
	92.8	0.7	2.3	77.1	0.4	Likely	0.2	0.1
	93.7	0.5	2.1	77.3	0.4	Likely	0.1	<0.1
							1.9	**1.0**
CPT-4	18.4	1.1	2.0	91.6	0.3	Likely	0.4	0.2
	21.3	1.0	2.3	111.1	0.4	Likely	0.3	0.2
	27.2	0.5	2.2	89.6	0.3	Likely	0.1	<0.1
	33.3	0.5	2.4	100.4	0.3	Likely	0.1	<0.1
	39.7	1.5	2.1	118.4	0.5	Likely	0.4	0.2
	78.1	0.3	2.2	50.7	0.3	Likely	0.1	<0.1

CPT Number	Depth to Top of Sand/Silt Layer (feet)	Layer Thickness (feet)	I_C	$*q_{C1N}$	Factor of Safety	Potential for Liquefaction	Estimated Total Settlement (in.)	Estimated Differential Settlement (in.)
CPT-4	101.4	0.5	1.8	102.8	0.7	Likely	0.1	<0.1
	114.5	0.3	2.1	102.8	0.7	Likely	0.1	<0.1
	127.1	0.3	2.2	97.4	0.7	Likely	0.1	<0.1
	128.2	0.7	2.1	90.6	0.6	Likely	0.2	0.1
							1.9	**1.0**
CPT-5	16.7	0.5	2.3	132.2	0.6	Likely	0.1	<0.1
	18.2	3.3	1.9	98.8	0.3	Likely	1.0	0.5
	22.1	0.7	2.1	109.4	0.4	Likely	0.2	0.1
	23.1	3.0	2.1	121.4	0.5	Likely	0.7	0.4
	26.4	0.3	2.3	88.7	0.3	Likely	0.1	<0.1
	27.7	1.6	2.2	82.1	0.3	Likely	0.6	0.3
	34.1	0.7	2.2	76.3	0.2	Likely	0.2	0.1
							2.9	**1.5**
CPT-6	20.5	0.3	2.0	133.3	0.6	Likely	<0.1	<0.1
	52.3	1.0	1.9	130.1	0.8	Likely	0.2	0.1
	62.5	1.3	2.3	108.3	0.6	Likely	0.4	0.2
	65.3	0.8	2.0	117.8	0.7	Likely	0.2	0.1
	66.3	2.6	2.0	102.1	0.6	Likely	0.7	0.4
	69.1	0.3	2.2	71.6	0.4	Likely	0.1	<0.1
	76.3	1.3	2.0	111.7	0.7	Likely	0.3	0.2
	100.7	0.7	2.3	112.9	0.8	Likely	0.1	<0.1
	118.4	0.6	2.1	103.0	0.7	Likely	0.2	0.1
	120.9	9.4	1.7	92.2	0.6	Likely	1.4	0.7
							3.7	**1.8**
CPT-7	24.1	0.3	2.4	50.2	0.2	Likely	0.2	0.1
	24.7	0.8	2.3	70.3	0.2	Likely	0.3	0.2
	37.6	1.6	2.1	106.0	0.4	Likely	0.5	0.3
	52.8	1.0	1.9	118.1	0.7	Likely	0.2	0.1
	63.3	5.1	1.8	114.1	0.7	Likely	1.2	0.6
	119.6	0.3	2.2	97.7	0.7	Likely	<0.1	<0.1
	126.3	0.7	2.1	100.1	0.7	Likely	0.2	0.1
						Total =	**2.7**	**1.4**

*CPT tip pressure corrected for overburden and fines content

The accepted method available to determine liquefaction settlement is applicable to only the upper 50 feet of soil. Because we do not have a method to accurately calculate the settlement below 50 feet, the liquefaction settlement calculated below 50 feet in Table 5 is an estimate based on the same method used for the upper 50 feet. In our opinion deep liquefaction could occur and might cause loss of soil strength but the settlement would likely be more uniform across the site. Based on our engineering judgment we have reduced the liquefaction settlement calculated for the approximately 9½ -foot-thick layer encountered in CPT-6 at 121 feet below the ground surface by half. By reducing the liquefaction settlement in CPT-6 the resulting liquefaction settlement across the site is more uniform and more representative of post earthquake settlement.

Our analyses indicate that several silt and sand layers theoretically can liquefy, resulting in about 1½ to 3¾ inches of total settlement, and ¾ to 1¾ inches of differential settlement. Estimates of volumetric change and settlement were determined by the Ishihara and Yoshimine (1990) method. As discussed in the SCEC report, differential movement for level ground, deep soil sites, will be on the order of half the total estimated settlement.

3.7.2 Potential for Ground Rupture/Sand Boils

The methods of analysis used to determine estimated total settlement assume that there is no possibility of surface ground rupture. In order for liquefaction induced sand boils or fissures to occur, the pore water pressure induced within the liquefied strata must exert a large enough force to break through the surface layer.

There is approximately 18 feet of non-liquefiable material overlying the liquefiable layers in CPT- 1, CPT-2, CPT-3, CPT-4, CPT-6, and CPT-7 at the site. Based on work by Youd and Garris (1995), it is our opinion that there is enough of a cap of non-liquefiable material to prevent ground rupture near CPT- 1, CPT-2, CPT-3, CPT-4, CPT-6, and CPT-7 at the site and that the above estimates of liquefaction induced settlement are reasonable.

Ground rupture is possible due to the potential shallow liquefaction for CPT-5 at Building B and Boring EB-8 at Building C due to the lack of a sufficient cap of non-liquefiable soil. If ground rupture and sand venting were to occur, significantly higher ground deformation may result. The total and differential liquefaction settlement estimates above are not appropriate for the shallow layers. The settlement estimates below a depth of 30 feet are reasonable. Please refer to the Conclusion Section of this report for recommendations regarding the ground rupture.

3.8 Seismically-Induced Differential Settlements

If near-surface soils vary in composition both vertically and laterally, strong earthquake shaking can cause non-uniform settlement of soil strata, resulting in movement of the near-surface soils. There is the potential for ground rupture near CPT-5 at the site. Provided the recommendations in the “Foundations” section of this report are followed, we judge the probability of significant differential compaction affecting the structures to be low.

3.9 Lateral Spreading

Lateral spreading typically occurs as a form of horizontal displacement of relatively flat-lying alluvial material toward an open or “free” face such as an open body of water, channel, or excavation. In soils this movement is generally due to failure along a weak plane, and may often be associated with liquefaction. As cracks develop within the weakened material, blocks of soil displace laterally towards the open face. Cracking and lateral movement may gradually propagate away from the face as blocks continue to break free. Generally, failure in this mode is analytically unpredictable, since it is difficult to determine where the first tension crack will occur.

The site is located between ¼ and ½ mile from the Guadalupe River, the shallow potentially liquefiable soil strata appear to be relatively discontinuous, and are between 16 and 28 feet below the existing ground surface. In addition, Buildings A and B and the parking garage will be supported on pile foundations founded well below the bottom of the river channel. Mitigation of the potentially liquefiable soils will likely have to be performed for the support of the floor slab for Building B and Building C.

3.10 Flooding

As shown on the February 19, 1986 Federal Emergency Management Agency (FEMA) "Flood Insurance Rate Map" (FIRM), this northwest portion of the site is within Zones AH, described as "Area of 100-year shallow flooding where depths are between one (1) and three (3) feet; base flood elevations are shown, but no flood hazard factors are determined." The base flood elevation shown is Elevation 27. The remaining portion of the site is within Zone D, described as "Areas of undetermined, but possible, flood hazards."

4.0 CONCLUSIONS AND DEVELOPMENT CONSIDERATIONS

4.1 Conclusions

From a geotechnical engineering viewpoint the proposed structures may be constructed as planned, in our opinion, provided the design is performed in accordance with the recommendations presented in this report.

The primary geotechnical and geologic concerns at the site are as follows:

- The potential for liquefaction-induced total and differential settlement of shallow and deep sand layers and potential ground rupture/sand boils of shallow liquefiable layers
- The presence of shallow ground water
- The presence of highly expansive soils blanketing the site
- Strong Seismic Shaking

For this report, we have prepared a brief description of the issues and presented typical approaches to manage potential concerns associated with the long-term performance of the development.

4.1.1 Liquefaction-Induced Settlements and Ground Rupture

Liquefaction-induced settlements and ground rupture are the primary concerns for this site. As discussed in the "Liquefaction" section of this report, several silt and sand layers theoretically can liquefy, resulting in about 1½ to 3¾ inches of total settlement in the buildings; we anticipate there will be additional settlement of the shallow sand layers in the southern portion on Building B (near CPT-5) due and Building C (near Boring EB-8) to ground rupture. If ground rupture and sand venting were to occur, significantly higher ground deformation could occur. Liquefied sands may vent through cracks in the surficial soils. The resulting sand boils will contribute to the magnitude of liquefaction induced settlement and could impact site improvements such as streets, sidewalks, or exterior concrete flatwork as well as the buildings.

It is our understanding that Tishman Speyer is considering deep foundations for the parking structure, Building A, and Building B and shallow foundations for Building C. We recommend that the pile caps for Buildings A, B and the parking garage be tied together with grade beams to provide increased lateral stiffness of the foundations, and either a mat foundation or ground improvement be performed at the Location of Building C. Detailed recommendations for deep foundations are presented in the "Earthwork" and "Foundations" sections of this report.

4.1.2 Shallow Ground Water

As discussed above, ground water is reportedly as shallow as 5 feet below the existing site grades. Our experience with similar sites in the vicinity indicates that shallow ground water could impact grading and underground construction.

4.1.3 High Expansion Potential of Surficial Soils

As discussed, highly expansive surficial soils were encountered at the site. Expansive soils can undergo significant volume change with changes in moisture content. They shrink and harden when dried and expand and soften when wetted. If structures are underlain by expansive soils it is important that foundation systems be capable of tolerating or resisting any potentially damaging soil movements. We recommend the slabs-on-grade have sufficient reinforcement and be supported on a layer of non-expansive fill. In addition, it is important to limit moisture changes in the surficial soils by using positive drainage away from buildings as well as limiting landscaping watering. To reduce the potential for damage to the planned at-grade sidewalks and flatwork, we recommend that they have sufficient reinforcement and be supported on a layer of non expansive fill as discussed later in this report.

4.1.4 Strong Seismic Shaking

We recommend that at a minimum, the proposed structures be designed in accordance with the seismic design criteria of the 2007 CBC. Site seismic coefficients are presented in the Foundation section below.

4.2 Plans, Specifications, and Construction Review

We recommend that our firm perform a plan review of the geotechnical aspects of the project design for general conformance with our recommendations. In addition, subsurface materials encountered in the relatively small diameter, widely spaced borings may vary significantly from other subsurface materials on the site. Therefore, we also recommend that a representative of our firm observe and test the geotechnical aspects of the project construction. This will allow us to form an opinion about the general conformance of the project plans and construction with our recommendations. In addition our observations during construction will enable us to note subsurface conditions that may vary from the conditions encountered during our investigation, and if needed, provide supplemental recommendations. For the above reasons, our geotechnical recommendations are contingent upon our firm providing geotechnical observation and testing services during construction.

5.0 EARTHWORK

5.1 Clearing and Site Preparation

The site should be cleared of all surface and subsurface improvements to be removed and deleterious materials including fills, flatwork, and debris. Abandonment of existing buried utilities is discussed below. We recommend that trees and shrubs designated to be removed should include the entire rootball and all roots larger that ½-inch in diameter. Depressions resulting from removal of trees and shrubs should be cleaned of loose soils and roots, and properly backfilled in accordance with the “Compaction” section of this report. Excavations extending below the planned finished site grades should be cleaned and backfilled with suitable material compacted as

recommended in the “Compaction” section of this report. We recommend that backfilling of holes or pits resulting from demolition and removal of buried structures be carried out under our observation and that backfill be tested during placement.

After clearing, any vegetated areas should be stripped to sufficient depth to remove all surface vegetation and topsoil containing greater than 3 percent organic matter by weight. At the time of our field investigation, we estimated that a stripping depth of approximately three inches would be required in vegetated areas. The actual stripping depth required depends on site usage prior to construction and should be established in the field by us at the time of construction. The stripped materials should be removed from the site or may be stockpiled for use in landscaped areas, if desired.

Alternatively, the site may be prepared for grading by mowing all surface vegetation so that only 1 to 2 inches of stubble remains. After removing the mowed vegetation from the site, the ground should be disked in two directions to a depth of at least 12 inches. In our opinion, this procedure should adequately mix the remaining organic root layer with the underlying soils prior to grading.

5.2 Removal of Existing Fill

Although fill was not encountered in during our exploration of the site, if fill is encountered during site grading it should be removed down to native soil. If the fill material meets the requirements in the “Material for Fill” section below, it may be reused as engineered fill. Side slopes of fill excavations in building and pavement areas should be sloped at inclinations no greater than 3:1 (horizontal:vertical) to minimize abrupt variations in fill thickness. All fill should be compacted in accordance with the recommendations for fill presented in the “Compaction” section of this report.

5.3 Abandoned Utilities

Abandoned utilities within the proposed building area should be removed in their entirety. As an alternative, it may be feasible to abandon (in-place) underground utilities within the proposed building area provided the utility does not conflict with new improvements, is completely grouted, and previous fills associated with the utility do not pose a risk to the structure. Existing underground utilities outside the proposed building area(s) should be removed or abandoned in-place by grouting or plugging the ends with concrete. The decision to abandon in-place versus removal should be based on the level of risk associated with the particular utility line.

It should be noted that fills associated with underground utilities abandoned in-place may have an increased potential for settlement, and partially grouted or plugged pipelines will have a potential risk of collapse that may result in ground settlement, soil piping, and leakage of pipeline constituents. The potential risks are relatively low for small diameter pipes (4 inches or less) and increasingly higher with increasing diameter.

5.4 Subgrade Preparation

After the site has been properly cleared, stripped, and necessary excavations have been made, exposed surface soils in those areas to receive fill, slabs-on-grade, foundations, or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the recommendations for fill presented in the “Compaction” section.

The finished compacted subgrade should be firm and non-yielding under the weight of compaction equipment. If the relative compaction of the subgrade is less than recommended or the surface of

the subgrade has significant yielding, then the area should be over-excavated and rebuilt or reworked until the subgrade conforms to our recommendations.

5.5 Material for Fill

All on-site soils below the stripped layer having an organic content of less than 3 percent by weight are suitable for use as fill at the site. In general, fill material should not contain rocks or lumps larger than 6 inches in greatest dimension, with 15 percent or less larger than 2½ inches in the greatest dimension. Stockpiles of on-site general fill can be used for grading outside of building and pavement areas.

Existing on-site fills if used as structural or non-expansive fill material should be inorganic and should have a Plasticity Index (PI) of 15 or less, which based on visual classification, they have. The fill should have sufficient binder to reduce the potential for sidewall caving of foundation and utility trenches. Confirmation testing of the PI values should be done prior to its use.

5.6 Compaction

All fill, as well as scarified surface soils in those areas to receive fill or slabs-on-grade, should be compacted to at least 90 percent relative compaction as determined by ASTM Test Designation D1557, latest edition, at a moisture content at least 1 percent over laboratory optimum, except for the native expansive clays. The native expansive clays should be compacted to between 87 and 92 percent relative compaction at a moisture content at least 3 percent over optimum. Fill should be placed in lifts no greater than 8 inches in uncompacted thickness. Each successive lift should be firm and non-yielding under the weight of construction equipment.

In pavement areas, the upper 6 inches of subgrade and full depth of aggregate base [and sub-base] should be compacted to at least 95 percent relative compaction (ASTM D1557, latest edition), except for the native clays, which should be compacted as noted above. Aggregate base and all import soils should be compacted at a moisture content near the laboratory optimum.

5.7 Wet Weather Conditions

It should be understood that earthwork such as fill placement and trench backfill may be very difficult during wet weather, especially for fill materials with a significant amount of clay. If the water content of the fill increases significantly above the optimum moisture content, the soils will become soft, yielding, and difficult to compact. Therefore, we recommend that earthwork be performed during periods of suitable weather conditions, such as the “summer” construction season.

There are several alternatives to facilitate fill placement and trench backfill if earthwork is performed during the wet winter season, and the moisture content of the fill materials increases significantly above optimum moisture.

- Scarify and air dry until the fill materials have a suitable moisture content for compaction
- Over-excavation the fill and replace with suitable on-site or import materials with an appropriate moisture content.
- Install a geo-synthetic (geotextile or geogrid) to reduce surface yielding and reinforce soft fill

- Chemically treat with quicklime (CaO), kiln-dust, or cement to reduce the moisture content and increase the strength of the fill

The implementation of these methods should be reviewed on a case-by-case basis so that a cost effective approach may be used for the specific conditions at the time of construction.

5.8 Trench Backfill

Bedding and pipe embedment materials to be used around underground utility pipes should be well graded sand or gravel conforming to the pipe manufacturer's recommendations and should be placed and compacted in accordance with project specifications, local requirements or governing jurisdiction. General fill to be used above pipe embedment materials should be placed and compacted in accordance with local requirements or the recommendations contained in this section, whichever is more stringent.

On-site soils may be used as general fill above pipe embedment materials provided they meet the requirements of the "Material for Fill" section of this report. General fill should be placed in lifts not exceeding 8 inches in uncompacted thickness and should be compacted to at least 90 percent relative compaction (ASTM D1557, latest edition) by mechanical means only. Water jetting of trench backfill should not be allowed. The upper 6 inches of general fill in all pavement areas subject to wheel loads should be compacted to at least 95 percent relative compaction.

Please note that utility trenches extending below a depth of about 5 to 7 below existing site grades may encounter ground water. If ground water is encountered in utility trench excavations, crushed rock may be used as pipe bedding (if approved by the local jurisdiction) in order to provide a stable working platform for utility installation and backfill. The crushed rock should be compacted by vibratory methods until no further volume reduction is observed. In addition re-use of excavated materials may be difficult due to high in-situ moisture contents.

Utility trenches located adjacent to footings should not extend below an imaginary 1:1 (horizontal:vertical) plane projected downward from the footing bearing surface to the bottom edge of the trench. Where utility trenches will cross beneath footing bearing planes, the footing concrete should be deepened to encase the pipe or the utility trench should be backfilled with sand/cement slurry or lean concrete within the foundation bearing plane.

Where relatively higher permeability sand or gravel backfill is used in trenches through lower permeability soils, we recommend that a cut-off plug of compacted clayey soil or a 2-sack cement/sand slurry be placed where such trenches enter the building and pavement areas. This would reduce the likelihood of water entering the trenches from the landscaped areas and seeping through the trench backfill into the building and pavement areas and coming into contact with expansive subgrade material.

5.9 Temporary Slopes and Trench Excavations

The contractor should be responsible for all temporary slopes and trenches excavated at the site and design of any required temporary shoring. Shoring, bracing, and benching should be performed by the contractor in accordance with the strictest governing safety standards.

5.10 Surface Drainage

Positive surface water drainage gradients (2 percent minimum) should be provided within 5 feet of the buildings to direct surface water away from foundations and slabs towards suitable discharge facilities. Ponding of surface water should not be allowed on or adjacent to structures, slabs-on-grade, or pavements. Roof runoff should be directed away from foundation and slabs-on-grade. Roof runoff should be carried at least 5 feet away from foundations and slabs in closed conduits and directed to suitable discharge facilities. Downspouts may discharge onto splash-blocks provided the area is covered with concrete slabs or asphalt concrete pavements.

5.11 Site Infiltration

Our borings indicate that highly plastic clays with low hydraulic conductivity blanket the site. Generally, the higher the plasticity, the lower the permeability and hydraulic conductivity. Therefore, we judge the site infiltration rate will be very low for any proposed site detention/retention facilities, especially when the soil is saturated. As discussed above, ground water was encountered at a shallow depth. The Regional Water Quality Control Board (RWQCB) requires that a minimum of 10 feet be maintained between the seasonal high ground water level and the bottom of any infiltration facility, likely requiring pre-treatment of pavement runoff.

5.11 Landscaping Considerations

As the near-surface soils are highly expansive, we recommend greatly restricting the amount of surface water infiltrating these soils near structures and slabs-on-grade. This may be accomplished by:

- Selecting landscaping that requires little or no watering, especially within 3 feet of structures, slabs-on-grade, or pavements,
- Using low flow rate sprinkler heads,
- Regulating the amount of water distributed to lawn or planter areas by installing timers on the sprinkler system,
- Providing surface grades to drain rainfall or landscape watering to appropriate collection systems and away from structures, slabs-on-grade, or pavements,
- Preventing water from draining toward or ponding near building foundations, slabs-on-grade, or pavements, and
- Avoiding open planting areas within 3 feet of the building perimeter.

We recommend that the landscape architect consider these items when developing the landscaping plans.

5.12 Construction Observation

A representative from our company should observe and test the geotechnical aspects of the grading and earthwork for general conformance with our recommendations including, site preparation, selection of fill materials, and the placement and compaction of fill. To facilitate your

construction schedule we request sufficient notification (48 hours) for site visits. The project plans and specifications should incorporate all recommendations contained in the text of this report.

6.0 FOUNDATIONS

6.1 2007 CBC Site Coefficients (CBC) Site Seismic Coefficients

Based on our borings and review of available geologic maps and alluvium thickness maps of Santa Clara County (Rogers and Williams, 1974), the site is underlain by stiff soils extending to depths on the order of 500 feet, which can be characterized as soil profile type S_D. Based on the above information and local seismic sources, the site may be characterized for design in accordance with Chapter 16 of the 2007 CBC using the information in Table 5 below.

Table 5. 2007 CBC Site Class and Site Seismic Coefficients

Categorization/Coefficient	Design Value
Site Class (Table 1613.5.2)	D
Mapped Spectral Response Acceleration for Short Period, S_s (Figure 1613.5(3))	1.5
Mapped Spectral Response Acceleration at 1-second Period, S_1 (Figure 1613.5(4))	0.6
Value of Site Coefficient, F_a (Table 1613.5.3(1))	1.0
Value of Site Coefficient, F_v (Table 1613.5.3(2))	1.5
S_{MS} (Equation 16-37)	1.5
S_{M1} (Equation 16-38)	0.9

6.2 Anticipated Loading Conditions

Preliminary structural loads for Buildings A and B, provided by Mr. Jason Louie of Middlebrook + Louie Structural Engineers, the project structural engineer, indicate dead plus live loads on the order of 1930 kips for typical interior columns and 1170 kips for typical exterior columns for Building A and 1930 kips for typical interior columns and 1170 kips for typical exterior columns for Building A.

7.0 DEEP FOUNDATIONS SUPPORTING BUILDINGS A, B, AND PARKING STRUCTURE

7.1 Driven Friction Piles

We recommend that the proposed office buildings (A and B) and parking structure be supported on driven, precast, prestressed concrete friction piles. We recommend a grid foundation structure to accommodate the potential movements in addition to the deep foundations. A conventional slabs-on-grade may be used in conjunction with a pile foundation for Building A; we recommend a structurally supported slab for Building B because of the potential for ground rupture; conventional slab-on-grade pavements or asphalt concrete pavement may be used in the parking structure.

7.1.1 Vertical Loads

Our exploration indicates there is no significantly thick or continuous dense sand layer that would provide end bearing support. Therefore, pile support is expected to come predominantly from

frictional support in the stiff clays. We computed allowable downward vertical capacities for 14-, and 16-inch-square concrete piles, the results of which are presented on Figures 5A and 5B. The indicated capacities are for dead plus live loads with a factor of safety of 2.0. For example, an allowable vertical capacity of 200 kips dead plus live loads would require a 85-foot-long, 14-inch-square pile or a 75-foot-long, 16-inch-square pile. Dead loads should not exceed two-thirds of the computed capacities. Uplift loads should also not exceed two-thirds of the computed downward capacities. The pile capacities shown on Figure 5A and 5B and uplift loads may be increased by one-third under transient loading, including wind and seismic.

Gross capacity of the piles should not exceed the ultimate structural capacity, which is estimated at 320 and 350 kips for typical 14- and 16-inch-square pretensioned, prestressed concrete piles, respectively (Santa Fe-Pomeroy, Inc.) for an allowable 200 kip vertical capacity. The structural engineer should apply an appropriate factor of safety to the structural capacity of the piles. We have assumed a base of pile cap at Elevation 22 feet, or 5 feet below ground (HMH, 2007) for our analysis. To effectively minimize pile group effects and reduction in individual pile capacity, piles should be located with a minimum center-to-center spacing of three times the pile width.

Based on the maximum allowable loads for a single pile on Figure 4, we estimate total static settlements of less than ¾-inch to mobilize allowable static capacities. Therefore, post-construction pile foundation static settlements of less than ½-inch should be anticipated. Seismic settlements due to liquefaction are approximately 1½-inches total and ½-inch differential for Building A, 1-inch total and ½-inch differential for Building B, and up to 3½-inches total and about 1¾ inch differential for the parking structure. Differential settlements are anticipated to occur over horizontal distances of the long dimensions of the structures.

7.1.2 Lateral Loads On Piles

Lateral load resistance for pile-supported structures may be developed through pile bending/soil interaction. The magnitude of the lateral load resistance is dependent upon many factors, including pile stiffness and embedment length, conditions of fixity at the pile cap, the physical properties of the surrounding soils, the tolerable top deflection and the yield moment capacity of the pile.

To estimate lateral capacities of piles, we used a computer program that models the soil response in the form of load-deflection (p-y) curves to estimate the capacity of the piles to resist the expected lateral loads. The lateral load characteristics for 14- and 16-inch-square, driven concrete piles with free head and fixed head conditions are presented below.

Table 6. Estimated Lateral Pile Response

Pile Size (inches)	Head Condition	Deflection (inches)	Maximum Shear Force (kips)	Maximum Moment (ft-kips)	Depth to Maximum Moment (ft)
14	Free	¼	12.1	40.9	6
		½	17.0	65.8	7
	Fixed	¼	25.9	100.9	Top of Pile
		½	35.2	155.2	
16	Free	¼	15.3	55.7	7
		½	21.2	87.8	7½
	Fixed	¼	31.4	136.0	Top of Pile
		½	42.5	207.9	

The results in the table above represent the probable response of the piles under short-term loading conditions and include no factor-of-safety. Suitable factors-of-safety should be selected on the basis of the type of loading. Pile stiffnesses (EI) of 1.4×10^{10} lb-in^2 and 2.4×10^{10} lb-in^2 have been assumed in our calculations of load deflection for the 14- and 16-inch piles, respectively. We assumed a minimum compressive strength of 6,000 pound per square inch for concrete modulus calculations. If pile stiffness varies by no more than 20 percent than that reported above, load deflection characteristics can be approximated by multiplying the deflection values by the ratio of the pile stiffness (EI). We should evaluate the response of piles with significantly different stiffness.

The above lateral load characteristics are for single piles and may not be characteristic of the lateral load capacity of piles in a group. Group effects may reduce the allowable lateral load for a given deflection. We recommend that a pile group efficiency of 0.75 be used for pile groups 3-by-3 or smaller. A group reduction would not be necessary for groups of 1 or 2 piles. For pile groups larger than 3-by-3, we recommend that we review the final pile group layout and structural loads to further evaluate the pile group efficiency under lateral loading.

7.1.3 Passive Resistance Against Pile Caps and Grade Beams

If desired, the passive resistance of soil against pile caps and grade beams poured neat against well-compacted engineered fill may be used for lateral resistance. We recommend that an allowable passive pressure based on an equivalent fluid pressure of 300 pounds per cubic foot be used in design.

7.1.4 WEAP Analysis

At a minimum, we recommend that the pile contractor have a wave equation analysis of piles (WEAP) performed to confirm compatibility and driveability of the pile driving system with the pile type and soil conditions at the site. We should review the WEAP results prior to mobilization of pile driving equipment to the site.

7.1.5 Indicator Piles

It has been our experience that uncertainties associated with production pile driving can be reduced considerably by implementing an indicator pile program. An indicator pile program will also provide a better means of confirming the limits of layers where high driving resistance may be encountered, and to more accurately estimate final pile lengths.

We recommend that at least five indicator piles be installed for each building before the final pile casting lengths have been selected. The indicator piles should be driven with the same equipment that will be used to drive the production piles. We should review or select the indicator pile locations when structural drawings are made available. The indicator pile cast lengths should be based on the design lengths required to meet the desired capacity, plus 5 feet. It is expected that some indicator piles may not be driven to their entire length and will require cutting to provide the desired butt elevation. Indicator piles can be used for support of the structure and, therefore, should be located appropriately. We also suggest that one or more spare piles be delivered to the site during the indicator program.

7.1.6 PDA Monitoring

If desired, a Pile Driving Analyzer (PDA) can be used during the indicator pile program to determine approximate pile capacities through dynamic testing. PDA monitoring may allow a reduction in production pile lengths and thus cost savings to the project. PDA monitoring should be performed during indicator driving and on selected piles for restrikes; restrikes should be performed no sooner than seven days after initial driving. *Please note that restrike testing more than one day after installation may significantly alter the contractor's sequencing. Therefore, if restrike testing is selected for this project, is should be clearly identified on the plans and specifications to avoid unexpected costly change-orders for out of sequence moves.* PDA monitoring would be especially beneficial for checking tensile stresses in the piles and for evaluating pile integrity on any piles suspected of being damaged during indicator or production driving. Piles designated for PDA monitoring during indicator pile installation should be at least 10 feet longer than design length so that the gauges are not driven into the ground.

7.1.7 Production Pile Installation

We recommend that a pile hammer capable of delivering a minimum rated driving energy of 60,000 foot-pounds be used. If indicator piles are installed, the same hammer should be used for both the indicator piles and the production piles. The pile contractor should perform wave equation analysis to confirm the compatibility and driveability of the pile driving system with the pile type and soil conditions at the site. We should review the wave equation results prior to mobilization of pile driving equipment to the site.

Since the piles are designed for skin friction support, they should be driven to the desired tip elevation. If difficult driving conditions are encountered, we should review the driving record and evaluate potential tip capacity to allow reduction in pile length. We may also recommend that a Pile Driving Analyzer (PDA) be used during production driving to determine approximate pile capacities through dynamic analyses. PDA monitoring would be especially beneficial for checking restrike capacities of any piles short of required tip elevation or for evaluating pile integrity on any piles suspected of being damaged during driving. We should observe all indicator and production pile installation on a full-time basis.

7.2 Displacement Augercast Piles

While less common in the Bay area, augercast piles have been successfully used for a project in downtown San Jose in similar soil conditions. Augercast piles are cast-in-place concrete piles that are drilled using a hollow-stem auger and pumping sand-cement grout through the bottom of the auger as the auger is retracted. Two types of augercast piles are available: APG piles, which like piers, remove the soil column and replace it with grout; and the APGD piles, which displace the soil prior to grout placement. We anticipate that displacement piles are

feasible so the drilling spoils generated would be minor. Augercast piles are a low noise and vibration installation compared to driven piles, and would not require pre-drilling through the thick sand layers. Various types of steel reinforcing including rebar cages or H-piles may be installed into the still wet grout after drilling. If you would like to explore this option we can contact a licensed design builder to discuss load capacities.

7.3 Building A and C Slab-on-Grade

The slab-on-grade for Buildings A and C should be at least 5 inches thick and have a minimum concrete strength of 3,000 psi. Due to the high expansion potential of the near surface soils, we recommend that the slab-on-grade be supported on at least 18 inches of non-expansive fill (NEF). The NEF should be compacted to at least 90 percent relative compaction (ASTM D1557, latest edition) and the crushed rock should be consolidated in-place to provide firm, uniform support for the slab-on-grade.

Post-construction cracking of concrete slabs-on-grade is inherent in any project, especially where soil expansion potential is high. In our opinion, consideration should be given toward a maximum control joint spacing of 10 to 15 feet in both directions for the interior slab-on-grade construction. Adequate slab reinforcement should be provided to satisfy the anticipated use and loading requirements.

Slab reinforcing should be provided in accordance with the anticipated use and loading of the slab. Prior to placing the base material, the building subgrade should be prepared in accordance with the recommendations presented in the “Subgrade Preparation” section of this report. Due to the high expansion potential of the clayey native soil, we recommend that the contractor take special measures to protect the subgrade from any inflow of water during construction, especially after the floor slab has been cast. Areas to receive special attention include slab joints and areas where building columns pass through the floor slab.

If desired to limit moisture rise through the at-grade slabs, the guidelines presented in the “Moisture Considerations” section of this report should be considered.

7.4 Building B Slab-on-Grade

Ground rupture is possible due to the shallow-depth liquefaction at CPT-5 in Building B. We recommend the slab-on-grade for Building B be designed as a structural slab capable of spanning unsupported between grade beams because the deep foundation system will not address potential ground rupture and seismic settlement between pile groups. If the slabs will be on grade, the NEF recommendations for Building A are also applicable to Building B.

If desired to limit moisture rise through the at-grade slabs, the guidelines presented in the “Moisture Considerations” section of this report should be considered.

7.5 Parking Structure Pavement

The at-grade parking structure pavement may be designed in accordance with the recommendations in the “Pavements” section of this report provided the owner understands that the deep foundation systems will not address seismic settlement between pile groups. Significant distress could occur to the pavements after strong seismic shaking, requiring repair. If significant repairs are not desired, the parking structure pavements should also be designed as structural

slabs capable of spanning unsupported between grade beams and pile groups. Please refer to the Pavement Section of this report for recommendations for pavement thickness.

7.6 Moisture Protection Considerations

Since the long-term performance of concrete slabs-on-grade depends to a large degree on good design, workmanship, and materials, the following general guidelines are presented for consideration by the developer, design team, and contractor. The purpose of these guidelines is to aid in producing a concrete slab of sufficient quality to allow successful installation of floor coverings and reduce the potential for floor covering failures due to moisture-related problems associated with slab-on-grade construction. These guidelines may be supplemented, as necessary, based on the specific project requirements.

- A minimum 10-mil thick vapor barrier meeting minimum ASTM E 1745, Class C requirements should be placed directly below the slab. The vapor barrier should extend to the edge of the slab-on-grade. At least 4 inches of free-draining gravel, such as ½- or ¾-inch crushed rock with no more than 5 percent passing the ASTM No. 200 sieve, should be placed below the vapor barrier to serve as a capillary break. The crushed rock should be consolidated in place with vibratory equipment. The vapor barrier should be sealed at all seams and penetrations in accordance with manufacturer's recommendations and ASTM E1643 requirements. The crushed rock may be considered as the upper 4 inches of the non-expansive fill requirement.

- The concrete water/cement ratio should not exceed 0.45. Midrange plasticizers could be used to facilitate concrete placement and workability.

- Water should not be added after initial batching, unless the slump of the concrete is less than specified, and the resulting water/cement ratio will not exceed 0.45.

- Polishing the concrete surface with metal trowels should not be permitted.

- All concrete surfaces to receive any type of floor covering should be moist cured for a minimum of seven days. Moist curing methods may include frequent sprinkling, or using coverings such as burlap, cotton mats, or carpet. The covering should be placed as soon as the concrete surface is firm enough to resist surface damage. The covering should be kept continuously wet and not allowed to dry out during the required curing period.

- Water vapor emission levels and pH should be determined before floor installation as required by the manufacturer of the floor covering materials. Measurements and calculations should be made according to ASTM F1869-98 and F710-98 protocol.

The guidelines presented above are based on information obtained from various technical sources, including the American Concrete Institute (ACI), and are intended to present information that can be used to reduce potential long-term impacts from slab moisture infiltration. It should be noted, the application of these guidelines does not affect the geotechnical aspects of the foundation performance.

8.0 BUILDING C FOUNDATIONS

Because of the potential for ground rupture as discussed above, a stiffer foundation system consisting of spread footings with interconnected grade beams is recommended for the building as described below. This foundation system may experience significant settlement should ground

failure occur which may require relevelling footings and floor slabs. A structurally-supported floor slab should also be considered.

8.1 Spread Footings

The Fitness Center/Community Center may be supported on conventional continuous and/or isolated spread footings bearing on natural, undisturbed soil or compacted fill. All footings should have a minimum width of 18 inches and footing bottoms should extend at least 24 inches below lowest adjacent finished grade. Lowest adjacent finished grade may be taken as the bottom of interior slab-on-grade or the finished exterior grade, excluding landscape topsoil, whichever is lower. Because of the high expansion potential of the near-surface soils, this relatively deeper footing is recommended to place bearing surfaces below the zone of significant moisture fluctuation in order to reduce the effects of heave or shrinkage. All footings should be connected by grade beams spanning in both directions, connecting all columns.

Footings constructed in accordance with the above recommendations would be capable of supporting maximum allowable bearing pressures of 2,000 pounds per square foot (psf) for dead loads, 3,000 psf for combined dead and live loads, and 4,000 psf for all loads including wind or seismic. These allowable bearing pressures are based upon factors of safety of 3.0, 2.0, and 1.5 for dead, dead plus live, and seismic loads, respectively.

These maximum allowable bearing pressures are net values; the weight of the footing may be neglected for design purposes. All footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing.

All continuous footings should be reinforced with top and bottom steel to provide structural continuity and to help span local irregularities. Footing excavations should be kept moist by regular sprinkling with water to prevent desiccation. It is essential that we observe the all footing excavations before reinforcing steel is placed.

8.2 Foundation Settlement

Structural loads for Fitness Center/Community Center were not available for our review at the time of our investigation. Therefore, we assumed typical interior column dead plus live loads on the order of 100 kips and perimeter wall dead plus live load on the order of 2 to 4 kips per foot. Based on these assumed loads and the maximum allowable bearing pressures recommended above, we estimate that total static footing settlement should be less than approximately 1 inch, with post-construction differential movement between adjacent columns of approximately ½- inch. We should be retained to review the final foundation plans and structural loads to verify the above settlement estimates.

In addition to the above static settlements, based on CPT-4, CPT-5, and CPT-7, which are closest to the Fitness Center, total seismic settlements due to liquefaction are approximately 1 to 2-inches and approximately 1-inch differential settlement across the building.

8.3 Lateral Loads

Lateral loads may be resisted by friction between the bottom of footings and the supporting subgrade. A maximum allowable frictional resistance of 0.25 may be used for design. In addition, lateral resistance may be provided by passive pressures acting against footings poured neat

against competent soil. We recommend that an allowable passive pressure based on an equivalent fluid pressure of 300 pounds per cubic foot (pcf) be used in design. The upper 12 inches of soil should be neglected when determining lateral passive resistance.

9.0 RETAINING WALLS

9.1 Lateral Earth Pressures

Any proposed retaining walls should be designed to resist lateral earth pressures from adjoining natural materials, backfill, and surcharge loads. Provided that adequate drainage is provided as recommended below, we recommend that walls restrained from movement at the top be designed to resist an equivalent fluid pressure of 45 pounds per cubic foot (pcf) plus a uniform pressure of 8H pounds per square foot, where H is the distance in feet between the bottom of the footing and the top of the retained soil. Restrained walls should also be designed to resist an additional uniform pressure equivalent to one-half of any surcharge loads applied at the surface. Any unrestrained retaining walls with adequate drainage should be designed to resist an equivalent fluid pressure of 45 pcf plus one-third of any surcharge loads.

The above lateral earth pressures assume level backfill conditions and sufficient drainage behind the walls to prevent build-up of hydrostatic pressure from surface water infiltration and/or a rise in the ground water level. If adequate drainage is not provided, we recommend an equivalent fluid pressure of 40 pcf be added to the values recommended above for both restrained and unrestrained walls. Damp proofing of the walls should be included in areas where wall moisture and efflorescence would be undesirable.

9.2 Drainage

Adequate drainage may be provided by a subdrain system behind the walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. The permeable backfill should extend at least 2 feet out from the wall and to within 2 feet of outside finished grade. Alternatively, ½- to ¾-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as TCMirafi 140N or equivalent. The upper 2 feet of wall backfill should consist of relatively impervious compacted on-site clayey soil. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or Enkadrain drainage matting may be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. The drainage panel should be connected to the perforated pipe at the base of the wall, or to some other closed or through-wall system. Miradrain panels should terminate 18 to 24 inches from final exterior grade. The Miradrain panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil.

9.3 Backfill

Where surface improvements will be located over the retaining wall backfill, backfill placed behind the walls should be compacted to at least 95 percent relative compaction using light compaction equipment. Where no surface improvements are planned, backfill should be compacted to at least 90 percent. If heavy compaction equipment is used, the walls should be temporarily braced.

9.4 Foundation

Retaining walls may be supported on a continuous spread footing designed in accordance with the recommendations presented in the “Footings” section of this report. Lateral load resistance for the walls may be developed in accordance with the recommendations presented in the “Lateral Loads” section.

9.5 Swimming Pool

The proposed pool should be designed to resist at-rest lateral earth pressures from adjoining natural materials and/or backfill as well as hydrostatic pressures and any surcharge loads. We recommend that undrained pool walls be designed to resist an at-rest earth pressure equivalent to a fluid pressure of 85 pounds per cubic foot (pcf) be used for design. Pool walls should be designed to resist an additional uniform pressure of one-half of any surcharge loads applied at the surface.

Based on our site exploration, we anticipate that the bottom of the pool excavation may be wet and unstable. Therefore, it may be necessary to over-excavate the pool bottom by 12 to 18 inches and to place crushed rock or baserock to provide a stable working base.

The pool should have pressure relief valve[s] incorporated into the base of the pool structure to relieve ground water pressure on the pool bottom when empty and prevent uplift of the pool. It would be desirable for the pool bottom to be supported on at least 4 inches of 3/8-inch pea gravel or Caltrans Class 2 Permeable Material to allow for pressure relief across the pool base. Alternatively, a larger crushed rock (up to 1-inch maximum size) may be used provided filter fabric such as Mirafi 140N or equivalent is placed between the rock and the bottom of the excavation.

Proper surface drainage should be provided about the pool deck to divert water to catch basins and other inlets for water to be carried away in closed drainpipes. Also, flexible bituminous caulking or equivalent should be applied at the juncture of the pool and deck to prevent infiltration of surface water into the native moderately expansive soils.

Should ground failure occur, some damage to the swimming pool should be expected requiring repairs to the pool and decking.

10.0 PAVEMENTS

Pavements located in the vicinity of Buildings B and C could experience distress if ground failures due to liquefaction occur. Repairs to the pavements including reworking the subgrade soil and patching or repaving some areas should be anticipated.

10.1 Asphalt Concrete

Previous R-value testing at the site indicated an R-value of 7. Because of the high plasticity and expansion potential of the surface soils, we judged an R-value of 5 to be applicable for design. Using estimated traffic indices for various pavement-loading requirements, we developed the following recommended pavement sections based on Procedure 608 of the Caltrans Highway Design Manual, presented in Table 7.

Table 7. Recommended Asphalt Concrete Pavement Design Alternatives
Pavement Components
Design R–Value = 5

General Traffic Condition	Design Traffic Index	Asphalt Concrete (Inches)	Aggregate Baserock* (Inches)	Total Thickness (Inches)
Automobile	4.0	2.5	8.0	10.5
Parking	4.5	2.5	10.0	12.5
Automobile	5.0	3.0	10.0	13.0
Parking Channel	5.5	3.0	12.0	15.0
Truck Access &	6.0	3.5	12.0	16.0
Parking Areas	6.5	4.0	14.0	18.0

*Caltrans Class 2 aggregate base; minimum R-value equal to 78.

The traffic indices used in our pavement design are considered reasonable values for the proposed development and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. Because the native soils at the site are moderately to highly expansive, some increased maintenance and reduction in pavement life can be expected. The traffic parameters used for design were selected based on engineering judgment and not on information furnished to us such as an equivalent wheel load analysis or a traffic study.

Because the full thickness of asphalt concrete is frequently not placed prior to construction traffic being allowed to use the streets (or parking lots), rutting and pavement failures can occur prior to project completion. To reduce this occurrence, we recommend that either the full design pavement section be placed prior to use by construction traffic, or a higher Traffic Index (TI) be specified where construction traffic will use the pavement.

In addition, it has been our experience that asphalt concrete pavements constructed over expansive soils and adjacent to non-irrigated open space areas may experience cracking parallel to the edge of the pavement. This is typically caused by seasonal shrinkage and swelling adjacent to non-irrigated edges of the pavement. The cracks typically occur within the first few years of construction, and are typically located within a few to several feet of the edge of the pavement. The cracks, if they occur, can be filled with a bituminous sealant. Otherwise, a moisture barrier would need to be installed to a depth of at least 24 inches to reduce the potential for shrinkage of the pavement subgrade soils.

10.2 Portland Cement Concrete Pavements

Recommendations for exterior Portland Cement Concrete (PCC) pavements are presented below in Table 8. Since the expected Average Daily Truck Traffic (ADTT) is not known at this time, we have provided alternatives for minimum pavement thickness. An allowable ADTT should be chosen that is greater than expected for the development.

Table 8. Recommended Minimum PCC Pavement Thickness

Allowable ADTT	Minimum PCC Pavement Thickness (inches)
0.8	5
13	5½
130	6

Our design is based on an R-value of 5 and a 28-day unconfined compressive strength for concrete of at least 3,500 pounds per square inch. In addition, our design assumes that pavements are restrained laterally by a concrete shoulder or curb and that all PCC pavements are underlain by at least 8 inches of Class 2 aggregate base. We recommend that adequate construction and control joints be used in design of the Portland Cement Concrete pavements to control the cracking inherent in this construction.

10.3 Pavement Cut-off

Because the native soils at the site are highly expansive, surface water infiltration beneath pavements could significantly reduce the pavement design life. While the amount of reduction in pavement life is difficult to quantify, in our opinion, the normal design life of 20 years may be reduced to less than 10 years. Therefore, long-term maintenance greater than normal may be required.

To limit the need for additional long-term maintenance, it would be beneficial to protect at-grade pavements from landscape water infiltration by means of a concrete cut-off wall, deepened curbs, “Deep-Root Moisture Barrier,” or equivalent. However, if reduced pavement life and greater than normal pavement maintenance are acceptable, the cut-off barrier may be eliminated. If desired to install pavement cut-off barriers, they should be considered where pavement areas lie downslope of any landscape areas that are to be sprinklered or irrigated, and should extend to a depth of at least 4 inches below the base rock layer.

10.4 Asphalt Concrete, Aggregate Base and Subgrade

Asphalt concrete and aggregate base should conform to and be placed in accordance with the requirements of Caltrans Standard Specifications, latest edition, except that ASTM Test Designation D1557 should be used to determine the relative compaction of the aggregate base. Pavement subgrade should be prepared and compacted as described in the “Earthwork” section of this report.

10.5 Exterior Sidewalks

We recommend that exterior concrete flatwork, such as patios or pedestrian walkways, be at least 4 inches thick and underlain by at least 12 inches of NEF or Class 2 aggregate base compacted to a minimum of 90 percent relative compaction in accordance with ASTM Test Method D1557, latest edition. We recommend that exterior slabs be isolated from adjacent foundations and that adequate construction and control joints be used in design of the concrete slabs to control cracking inherent in concrete construction. If sidewalks are subject to wheel loads, they should be designed in accordance with the “Exterior Portland Cement Concrete Pavements” section of this report. As

with the pavements, if liquefaction-induced ground failures occur adjacent to Buildings B and C, some damage to the concrete flatwork should be anticipated requiring reworking the subgrade soil and patching or reconstructing portions of the flatwork.

11.0 LIMITATIONS

This report has been prepared for the sole use of Tishman Speyer, specifically for design of the Phase 1 of the Campus at North First Street project in San Jose, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in the San Francisco Bay Area at the time this report was written. No other warranty, expressed or implied, is made or should be inferred.

The opinions, conclusions and recommendations contained in this report are based upon the information obtained from our investigation, which includes data from widely separated discrete locations, visual observations from our site reconnaissance, and review of other geotechnical data provided to us, along with local experience and engineering judgment. The recommendations presented in this report are based on the assumption that soil and geologic conditions at or between borings do not deviate substantially from those encountered or extrapolated from the information collected during our investigation. We are not responsible for the data presented by others.

We should be retained to review the geotechnical aspects of the final plans and specifications for conformance with our recommendations. The recommendations provided in this report are based on the assumption that we will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, TRC cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of TRC's report by others. Furthermore, TRC will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services and/or at the time another consultant is retained for follow up service to this report.

The opinions presented in this report are valid as of the present date for the property evaluated. Changes in the condition of the property will likely occur with the passage of time due to natural processes and/or the works of man. In addition, changes in applicable standards of practice can occur as a result of legislation and/or the broadening of knowledge. Furthermore, geotechnical issues may arise that were not apparent at the time of our investigation. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of three years, nor should it be used, or is it applicable, for any other properties.

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APPENDIX A

FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem auger drilling and Cone Penetration Test (CPT) equipment. Eight 8-inch-diameter exploratory borings were drilled on October 8 to 12, 2007, to a maximum depth of 100 feet; seven CPTs were advanced on October 18 and 19, 2007, to a maximum depth of 130 feet. We performed three additional rotary wash borings next to previously performed CPT-1, CPT-3, and CPT-4 on November 27, 2007 to collect samples in the liquefiable sand layers. The approximate locations of the exploratory borings and CPTs are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). The logs of the borings and CPTs, as well as a key to the classification of the soil and CPT interpretations, are included as part of this appendix.

The locations of borings and CPTs were approximately determined by pacing from existing site boundaries. Elevations of the borings were not determined. The locations of the borings should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. Penetration resistance blow counts were obtained by dropping a 140-pound hammer 30 inches. Modified California 2.5-inch I.D. samples and Standard Penetration Test (SPT) 2-inch O.D. samples were obtained by driving the samplers 18 inches and recording the number of hammer blows for each 6 inches of penetration. Relatively undisturbed samples were also obtained with 2.875-inch I.D. Shelby Tube sampler which were hydraulically pushed. Unless otherwise indicated, the blows per foot recorded on the boring logs represent the accumulated number of blows required to drive the samplers the last two 6-inch increments. When using the SPT sampler, the last two 6-inch increments is the uncorrected Standard Penetration Test measured blow count. The various samplers are denoted at the appropriate depth on the boring logs and symbolized as shown on Figure A-1.

Field tests included an evaluation of the undrained shear strength of soil samples using a Torvane device, and the unconfined compressive strength of the soil samples using a pocket penetrometer device. The results of these tests are presented on the individual boring logs at the appropriate sample depths.

The attached boring logs and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

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APPENDIX B

LABORATORY PROGRAM

The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site and to aid in verifying soil classification.

Moisture Content: The natural water content was determined (ASTM D2216) on 139 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on 108 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Plasticity Index: Plasticity Index determinations (ASTM D4318) were performed on two samples of the subsurface soils to measure the range of water contents over which these materials exhibit plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of these tests are presented on Figure B-1 and on the logs of the borings at the appropriate sample depths.

Sieve and Hydrometer Analyses: Gradation and washed sieve analyses (ASTM D422 and D2217) were performed on four samples of the subsurface soils to aid in soil classification. Results of these tests are included in this appendix.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on 12 samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Unconsolidated Undrained Triaxial Shear Test: Two Unconsolidated Undrained Triaxial Shear Tests to find the undrained shear strength of these clayey samples. Results of these tests are shown on the logs at the appropriate depths and included as part of this appendix.

Consolidation: Consolidation tests (ASTM D2435) were performed on two undisturbed samples of the subsurface clayey soils to assist in evaluating the compressibility properties of these soils. Results of the consolidation tests are presented graphically on Figures ___.

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