REPORT OF GEOTECHNICAL CONSULTATION
PROPOSED HIGH-RISE OFFICE BUILDING AND RETAIL DEVELOPMENT

2000 AVENUE OF THE STARS
CENTURY CITY DISTRICT-LOS ANGELES, CALIFORNIA

Prepared for:

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Los Angeles, California

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Los Angeles, California

October 5, 2001

Project 70131-1-0242

WORKING DRAFT

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SUMMARY

We have completed our geotechnical consultation for the proposed redevelopment of the existing entertainment and retail complex at 2000 Avenue of the Arts in the Century City District of Los Angeles, California. We understand that the existing performing arts theater, movie theaters, and retail and office areas at the site will be demolished for the construction of a 15-story office building and two adjacent retail structures.

The soil conditions at the project site were previously explored by drilling 35 borings to depths of 6 to 135 feet below the pre-development ground surface at the locations shown in Figure 1. The natural soils below the lowest parking level consist primarily of fine sand with varying amounts of gravel and cobbles. These soils are predominantly dense to very dense. Although ground water was not encountered within the project site, water seepage was observed in two previous borings.

We understand that the new planned retail structures will be supported on existing columns. If the new column loads do not exceed the loads imposed by the existing structures, no modifications to the foundations are necessary. For the proposed high-rise office building, detailed supplementary settlement analysis will be required prior to finalizing the foundation plan.

The majority of these new columns for the planned high-rise building may be supported on new and enlarged spread footings established at the lowest level of the parking structure. In some areas, several spread footings may have to be combined into partial mat foundations to accommodate the design foundation loads. Shored excavations may be required to construct new footings and to enlarge existing footings adjacent to and within ventilation pits and tunnels.

Per the structural engineer's request, we have also developed recommendations for pile foundations for support of selected new columns and/or underpin selected existing spread footings. It may be possible to support new columns adjacent to pits and tunnels on pile foundations to avoid excavations and shoring. Similarly, existing columns adjacent to new proposed columns may be left intact if the new column is supported on pile foundations.

1.0 SCOPE

This report provides geotechnical design information for the proposed redevelopment. The locations of the proposed high-rise office building and retail structures, existing buildings, adjacent streets, and our prior exploration borings are shown in Figure 1, Plot Plan.

We have previously performed four geotechnical investigations at the project site. The results were submitted in reports dated May 10, 1967 (Job No. A-67065), July 2, 1968 (Job No. A-67065-B), July 19, 1969 (Job No. A-69036), and October 2, 1987 (Job No. A-87231). The recommendations presented in this report were developed in using geotechnical information obtained during our previous investigations.

Our current study was authorized to evaluate the characteristics of the soils at the site of the proposed redevelopment, and to provide recommendations for design of foundations and walls below grade, for floor slab support, for shoring, and for excavations and grading. We performed the following main tasks:

- review of prior subsurface explorations to determine the nature and stratigraphy of the subsurface soils and the ground water conditions,
- a geologic-seismic hazards evaluation, including an evaluation of the liquefaction potential of the soils underlying the site,
- a ground motion study to develop site-specific seismic response spectra, and
- engineering evaluation of the geotechnical data to develop recommendations for design of foundations and walls below grade, for floor slab support, and for earthwork for the proposed redevelopment.

The assessment of general site environmental conditions for the presence of contaminants in the soils and groundwater of the site was beyond the scope of this consultation. We understand that Envicom Corporation is performing the Environmental Impact Report (EIR) for the project. Environmental concerns identified in the EIR should be addressed in the design and construction of the project.

Our recommendations are based on the results of our previous field explorations, laboratory tests, and appropriate engineering analyses. The results of the previous field explorations and laboratory tests, which form the basis of our recommendations, are presented in the Appendix.

Our professional services have been performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, express or implied, is made as to the professional advice included in this report. This report has been prepared for Trammel Crow So. Cal., Inc. and their design consultants to be used solely in the design of the proposed redevelopment of 2000 Avenue of the Stars in the Century City District of Los Angeles, California. The report has not been prepared for use by other parties, and may not contain sufficient information for purpose of other parties or other uses.

2.0 PROJECT DESCRIPTION

We understand that the existing movie theater and performing arts theater at the project site (2000 and 2020 Avenue of the Stars) will be demolished for the construction of a new proposed high-rise office building and two adjacent retail structures. The proposed high-rise building will be of steel-frame construction and have 13 stories of office space over two levels of retail space. The adjacent retail structures are expected to be of concrete construction and be up to three stories high. An existing six-level parking structure, which will remain in place, underlies the project site. The parking structure and existing overlying structures are supported on spread footings established below the lowest parking level.

The proposed high-rise building will be supported on new columns extending through the existing parking structure. Based on our review of a preliminary foundation plan, typical dead plus live column loads appear to range from about 4,800 kips to 5,500 kips. The majority of these new columns may be supported on new and enlarged spread footings established at the lowest level of the parking structure. However, adjacent to utility trenches and pits and existing columns, we understand that pile foundations may be used to support the new columns and/or underpin existing foundations.

The proposed retail structures will be supported on existing columns. We were informed by Mr. Schindler of John A. Martin & Associates, Structural Engineers, that new column loads will not exceed the loads previously imposed on the existing columns by the structures that will be demolished.

3.0 SITE CONDITIONS

The project site is located in the Century City District of Los Angeles, California. It is located on the western half of a lot bounded by Avenue of the Stars to the west, Olympic Avenue to the south, Constellation Boulevard to the north, and Century Park East to the east. The project site is currently occupied by six-story buildings used for office and retail space, including a movie theater and performing arts theater. Two existing high-rise office buildings are located on the eastern half of the lot. A six-level subterranean parking structure underlies the entire lot.

The ground surface adjacent to the site is relatively level but slopes gently downward to the northeast. The concourse level of the project site ('ground" or "at-grade" floor of the existing six-story buildings) is approximately at Elevation 280 feet. The floor of the lowest subterranean parking level is approximately at Elevation to 215 feet.

Various underground utility trenches as well as ventilation pits and tunnels are located below the lowest parking level. We observed two pits that are used as ventilation rooms. The floor of these ventilation pits appeared to be at about 15 feet below the lowest parking level slab. One ventilation pit is located near the northwest corner of the planned high-rise building and is approximately 30 feet by 40 feet in plan view. The other ventilation pit is approximately 40 feet by 50 feet in plan view and is located near the middle of the eastern perimeter wall of the planned high-rise building.

Concrete tunnels that function as ventilation ducts extend from these ventilation pits to air intake shaft, stairwells, and elevator wells. These ventilation tunnels are about 10 feet below the lowest parking level slab. We also observed evidence of utility lines such as for chilled and hot water buried in trenches below the lowest parking level slab.

We observed evidence of water seepage through the west wall (parallel to Avenue of the Stars) and east wall (parallel to Century Park East) of the lowest parking level. These walls do not have subdrain systems installed behind them. A few sections of the west wall were discolored due to the seepage and weepholes had been installed to drain selected sections of this wall. A large portion of the east wall is discolored due to water seepage appears to have been previously patched and covered with sealant.

We observed evidence of floor slab heave at the lowest parking level. No evidence of water seepage through the floor slab was observed. The floor slab has an underlying sudrain system.

4.0 EXPLORATIONS AND LABORATORY TESTS

The soil conditions at the project site were previously explored by drilling 35 borings to depths of 6 to 135 feet below the pre-development ground surface at the locations shown in Figure 1. Details of the explorations and the logs of the borings are presented in the Appendix.

Laboratory tests were previously performed on selected samples obtained from the borings to aid in the classification of the soils and to determine the pertinent engineering properties of the foundation soils. The following tests were performed:

- moisture content and dry density determinations,
- direct shear,
- consolidation, and
- Expansion Index.

All testing was performed in general accordance with applicable ASTM specifications at the time the tests were performed. Details of the laboratory testing program and test results are presented in the Appendix.

5.0 SOIL CONDITIONS

The natural soils below the lowest parking level consist primarily of fine sand with varying amounts of gravel and cobbles. These soils are predominantly dense to very dense. Hard, cemented layers, up to five feet in thickness were encountered at various depths below the lowest parking level. Some of the previous borings were terminated due to hardness of the cemented layers.

Ground water was not encountered in the previous borings at the project site drilled to a maximum depth of approximately 45 feet below the lowest parking level. Ground water was also not encountered in the previous borings at the adjacent site of the Plaza Towers drilled to a maximum depth of 105 feet below the lowest parking level.

Although ground water was not encountered, water seepage was observed in several borings drilled within the project site and the adjacent Plaza Towers site. Within the project site, seepage was observed in two previous borings, approximately 5 feet above the bottom of the lowest parking level at Boring 23 from Job No. A-67065-B and at approximately 16 feet below ground surface in Boring 9 from Job No. A-67065. Water seepage was observed mostly near and above the lowest parking level in several borings at the adjacent Plaza Towers site. Although water seepage below the lowest parking level was not noted in most of the previous borings, the majority of borings extending to such depths were drilled using drilling mud that makes it difficult to establish ground water levels and areas of seepage.

6.0 GEOLOGY

6.1 GEOLOGIC SETTING

The site is located on the northern portion of the Coastal Plain of Los Angeles County on a topographic rise between the City of Beverly Hills and the Westwood District of the City of Los Angeles. This topographic feature is generally referred to as the Beverly Hills (California Department of Water Resources, 1961). These hills represent the northern limit of the Newport-Inglewood uplift or fault zone that extends southeasterly from the Beverly Hills to offshore of Newport Beach. Regionally, the site is in the Peninsular Ranges geomorphic province that is characterized by elongate northwest-trending mountain ridges separated by straight-sided sediment-filled valleys. The northwest trend is further reflected in the direction of the dominant geologic structural features of the province that are northwest to west-northwest trending folds and faults, such as the nearby Newport-Inglewood fault zone located 1.8 kilometers to the east of the site.

The site is within the limits of Beverly Hills Oil Field (California Division of Oil, Gas, and Geothermal Resources, 1996). The Beverly Hills dome, which is a small fold developed at depth in Tertiary age sediments, is the northernmost anticlinal structure of the Newport-Inglewood uplift. The dome is apparently not reflected in overlying sediments of Pleistocene age. The limited production of the oil field is from a Mio-Pliocene age zone (at a depth of approximately 900 to 1,200 meters).

The relationship of the site to local geologic features is depicted in Figure 2, Local Geology, and the faults in the vicinity of the site are shown in Figure 3, Regional Faults. Figure 4, Regional Seismicity, shows the locations of major faults and earthquake epicenters in Southern California.

6.2 GEOLOGIC MATERIALS

Thirty-seven (37) borings were drilled at the site as part of our prior investigation in 1967 and 1969. Based on the materials encountered in our prior borings, artificial fill soils had mantled the site in 1967 and 1969. These fill soils were removed during construction of the existing theater buildings and parking structure. The project site is currently primarily underlain by late Pleistocene age older alluvial deposits (California Division of Mines and Geology, 1998). As encountered in our previous borings at the site, the alluvial deposits below the lowest parking level are primarily fine sand with varying amounts of gravel and cobbles. In the vicinity of the site, the late Pleistocene age alluvial deposits are between 15 and 26 meters (50 and 85 feet) thick, and are underlain by approximately 200 meters (650 feet) of early Pleistocene age sediments of the San Pedro Formation. The San Pedro Formation sediments are underlain by Tertiary age sedimentary rocks that are estimated to extend to a depth of approximately 4,000 meters (13,000 feet) beneath the site (Yerkes, 1965; California Department of Water Resources, 1961; Poland, 1959).

6.3 GROUND WATER

The site is located in Section 26 of Township 1 South, Range 15 West within the Santa Monica Hydrologic Subarea of the Los Angeles County Coastal Plain Hydrologic Subunit. Current ground-water level information for the site and the surrounding area is limited. The Los Angeles County Flood Control District's ground-water monitoring program ceased in about 1978. However, in recent years, the county has begun monitoring a select number of observation wells. Based on a review of available records, there are no nearby observation wells in the site vicinity to provide current ground-water level data.

Ground water was not encountered in the previous borings at the project site drilled to a maximum depth of approximately 45 feet below the lowest parking level. Ground water was also not encountered in the previous borings at the adjacent site of the Plaza Towers drilled to a maximum depth of 105 feet below the lowest parking level. Although ground water was not encountered, water seepage was observed in several borings drilled within the project site and the adjacent Plaza Towers site. Within the project site, seepage was observed in two previous borings, approximately

5 feet above the bottom of the lowest parking level at Boring 23 from Job No. A-67065-B and at approximately 16 feet below ground surface in Boring 9 from Job No. A-67065.

According to the California Division of Mines and Geology (1998), the historic high ground-water level beneath the site ranges in depth from 30 to 40 feet beneath the existing ground surface. The current ground-water levels beneath the site could be different than those encountered during our previous investigations in 1967 and 1969. Additionally, zones of perched water could occur locally at higher elevations within the alluvial deposits beneath the site.

6.4 FAULTS

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Division of Mines and Geology (CDMG) for the Alquist-Priolo Earthquake Fault Zoning Program (Hart, 1997). By definition, an active fault is one that has had displacement within Holocene time (about the last 11,000 years). A potentially active fault is a fault that has demonstrated displacement of Quaternary age deposits (last 1.6 million years). Inactive faults have not moved in the last 1.6 million years. A list of nearby active faults and the distance in kilometers between the site and the nearest point on the fault, the maximum magnitude, and the slip rate for the fault is given in Table 1. A similar list for potentially active faults is presented in Table 2. The surface faults in the vicinity of the site are shown in Figures 2 and 3.

Active Faults

<u>Santa Monica Fault</u>

The closest active fault to the site is the Santa Monica fault located approximately 460 meters to the north (Pratt et al., 1998). The Santa Monica fault is the western segment of the Santa Monica-Hollywood fault zone which trends east-west from the Santa Monica coastline on the west to the Hollywood area on the east. In the Santa Monica area, the Santa Monica fault splays into two segments, the North Branch and the South Branch. Several investigators (Dolan et al., 2000a; Dolan et al., 1997; Hummon et al., 1992; Dolan and Sieh, 1992; and Crook and Proctor, 1992) have indicated that the fault is active, based on geomorphic evidence and fault trenching studies. Also, several recent studies indicate that the Santa Monica fault does not extend east of the northerly extension of the Newport-Inglewood fault zone or the West Beverly Hills Lineament of Dolan and Sieh (1997, 1992). An Alquist-Priolo Earthquake Fault Zone has not been established for the Santa Monica fault because of the absence of well-defined fault traces. However, the Santa Monica fault is considered active by the State Geologist.

Newport-Inglewood Fault Zone

The active Inglewood fault of the Newport-Inglewood fault zone is located approximately 1.8 kilometers east of the site. This fault zone is composed of a series of discontinuous northwest-trending en echelon faults extending from Ballona Gap southeastward to the area offshore of Newport Beach. This zone is reflected at the surface by a line of geomorphically young anticlinal hills and mesas formed by the folding and faulting of a thick sequence of Pleistocene age sediments and Tertiary age sedimentary rocks (Barrows, 1974). Fault-plane solutions for 39 small earthquakes (between 1977 and 1985) show mostly strike-slip faulting with some reverse faulting along the north segment (north of Dominguez Hills) and some normal faulting along the south segment (south of Dominguez Hills to Newport Beach) (Hauksson, 1987). Investigations by Law/Crandall (1993) in the Huntington Beach area indicate that the north branch segment of the Newport-Inglewood fault zone offsets Holocene age alluvial deposits in the vicinity of the Santa Ana River.

Hollywood Fault

The active Hollywood fault is located approximately 3.2 kilometers north of the site. This fault trends east-west along the base of the Santa Monica Mountains from the West Beverly Hills Lineament in the West Hollywood-Beverly Hills area (Dolan and Sieh, 1992) to the Los Feliz area of Los Angeles. The fault is a ground-water barrier within Holocene sediments (Converse et al., 1981). Scarps 1.8 to 2.7 meters high in Holocene flood plain deposits have been suggested along the fault trace in the Atwater area (Weber et al. 1980). Studies by several investigators (Dolan et al., 2000b; Dolan et al., 1997; Dolan and Sieh, 1992; and Crook and Proctor, 1992) have indicated that the fault is active, based on geomorphic evidence, stratigraphic correlation between exploratory borings, and fault trenching studies. Additionally, recent investigations performed in the Hollywood area by Law/Crandall (2000) have demonstrated that Holocene age alluvial sediments have been offset by several strands of the Hollywood fault. An Alquist-Priolo Earthquake Fault Zone has not been established for the Hollywood fault. However, the Hollywood fault is considered active by the State Geologist. Also, the City of Los Angeles considers the Hollywood fault active for planning purposes.

San Andreas Fault Zone

The active San Andreas fault zone is located about 61 kilometers northeast of the site. This fault zone, California's most prominent geological feature, trends generally northwest for almost the entire length of the state. The southern segment, closest to the site, is approximately 450 kilometers long and extends from the Mexican Border to the Transverse Ranges west of Tejon Pass. Wallace (1968) estimated the recurrence interval for a magnitude 8.0 earthquake along the entire fault zone to be between 50 and 200 years. Sieh (1984) estimated a recurrence interval of 140 to 200 years. The 1857 Fort Tejon earthquake was the last major earthquake along the San Andreas fault zone in Southern California.

Blind Thrust Fault Zones

Northridge Thrust

The Northridge Thrust, as defined by Petersen et al. (1996), is an inferred deep thrust fault that is considered the eastern extension of the Oak Ridge fault. The Northridge Thrust is located beneath the majority of the San Fernando Valley and is believed to be the causative fault of the January 17, 1994 Northridge earthquake. This thrust fault is not exposed at the surface and does not present a potential surface fault rupture hazard. However, the Northridge Thrust is an active feature that can generate future earthquakes. The vertical surface projection of the Northridge Thrust is approximately 10 kilometers northwest of the site at the closest point. Petersen et al. (1996) estimates an average slip rate of 1.5 mm/yr. and a maximum credible earthquake of magnitude 6.9 for the Northridge Thrust.

Compton-Los Alamitos Thrust

The Compton-Los Alamitos Thrust, as defined by Dolan et al. (1995), is an inferred blind thrust fault located within the south-central portion of the Los Angeles Basin. The closest edge of the vertical surface projection of the buried thrust fault is located about 12 kilometers southeast of the site. This deep buried thrust fault is suggested to extend over 80 kilometers from the Santa Monica Bay coastline southeast into northwestern Orange County. The Compton-Los Alamitos Thrust may connect with the Elysian Park Thrust (to the northeast) along a detachment fault below Los Angeles. Like other blind thrust faults in the Los Angeles area, the Compton-Los Alamitos Thrust is not exposed at the surface and does not present a potential surface rupture hazard. However, the Compton-Los Alamitos Thrust should be considered an active feature capable of generating future earthquakes. An average slip rate of 1.5 mm/yr and a maximum credible earthquake of magnitude 6.8 are estimated by Petersen et al. (1996) for the Compton-Los Alamitos Thrust.

Elysian Park Thrust

The Elysian Park Thrust, previously defined by Hauksson (1990) as the Elysian Park Fold and Thrust Belt, was postulated to extend northwesterly from the Santa Ana Mountains to the Santa Monica Mountains, extending westerly and paralleling the Santa Monica-Hollywood and Malibu Coast faults. The Elysian Park Thrust is now believed to be smaller in size, only underlying the central Los Angeles Basin (Petersen et al., 1996). The vertical surface projection of the Elysian Park Thrust is about 18½ kilometers east-southeast of the site at its closest point. Like other blind thrust faults in the Los Angeles area, the Elysian Park Thrust is not exposed at the surface and does not present a potential surface rupture hazard; however, the Elysian Park Thrust should be considered an active feature capable of generating future earthquakes. An average slip rate of 1.5 mm/yr and a maximum magnitude of 6.7 are estimated by Petersen et al. (1996) for the Elysian Park Thrust.

Potentially Active Faults

Overland Fault

The closest potentially active fault to the site is the Overland fault located approximately 1.8 kilometers to the west. The Overland fault trends northwest between the Charnock fault and the Newport-Inglewood fault zone. The fault extends from the northwest flank of the Baldwin Hills to Santa Monica Boulevard in the vicinity of Overland Avenue. Based on water level measurements, displacement along the fault is believed to be vertical, with an offset of about 9 meters (Poland, 1959). The west side of the fault has apparently moved downward, relative to the east side, forming a graben between the Charnock and Overland faults. However, there is no evidence that this fault has offset late Pleistocene or Holocene age alluvial deposits (County of Los Angeles Seismic Safety Element, 1990). Ziony and Jones (1989) indicate that the fault is potentially active (no displacement of Holocene age alluvium). Additionally, the State Geologist considers this fault to be potentially active (Jennings, 1994).

Charnock Fault

The potentially active Charnock fault is located approximately 5.4 kilometers south-southwest of the site. The Charnock fault trends northwest-southeast, subparallel to the trend of the Newport-Inglewood fault zone and the Overland fault. Differential water levels across the fault occur in the early Pleistocene age San Pedro Formation. However, there is no evidence that this fault has offset late Pleistocene or Holocene age alluvial deposits (County of Los Angeles Seismic Safety Element. 1990). Ziony and Jones (1989) indicate that the fault is potentially active (no displacement of Holocene age alluvium). Additionally, the State Geologist considers this fault to be potentially active (Jennings, 1994).

MacArthur Park Fault

The potentially active MacArthur Park fault is located about 10½ kilometers east-northeast of the site. The fault, inferred west of downtown Los Angeles, has been located based on south-facing scarps, truncated drainages, and other geomorphic features (Dolan and Sieh, 1993). The fault is approximately 8 kilometers long, extending northwest from the Pershing Square area in downtown Los Angeles through MacArthur Park to the Paramount Studios area in Hollywood. Current information suggests the fault is potentially active.

6.5 GEOLOGIC-SEISMIC HAZARDS

Fault Rupture

The site is not within a currently established Alquist-Priolo Earthquake Fault Zone for surface fault rupture hazards. The closest Alquist-Priolo Earthquake Fault Zone, established for a portion of the Inglewood fault of the Newport-Inglewood fault zone, is located approximately 4.4 kilometers to the southeast of the site. Based on the available geologic data, active or potentially active faults with the potential for surface fault rupture are not known to be located directly beneath or projecting toward the site. Therefore, the potential for surface rupture due to fault plane displacement propagating to the surface at the site during the design life of the project is considered low.

Seismicity

Earthquake Catalog Data

The seismicity of the region surrounding the site was determined from research of an electronic database of seismic data (Southern California Seismographic Network, 2001). This database includes earthquake data compiled by the California Institute of Technology for 1932 to 2000 and data for 1812 to 1931 compiled by Richter and the U.S. National Oceanic Atmospheric Administration (NOAA). The search for earthquakes that occurred within 100 kilometers of the site indicates that 400 earthquakes of Richter magnitude 4.0 and greater occurred between 1932 and 2000: no earthquakes of magnitude 6.0 or greater occurred between 1906 and 1931; and one earthquake of magnitude 7.0 or greater occurred between 1812 and 1905. A list of these earthquakes is presented as Table 3. Epicenters of moderate and major earthquakes (greater than magnitude 6.0) are shown in Figure 4.

The information for each earthquake includes date and time in Greenwich Civil Time (GCT), location of the epicenter in latitude and longitude, quality of epicentral determination (Q), depth in kilometers, distance from the site in kilometers, and magnitude. Where a depth of 0.0 is given, the solution was based on an assumed 16-kilometer focal depth. The explanation of the letter code for the quality factor of the data is presented on the first page of the table.

Historic Earthquakes

A number of earthquakes of moderate to major magnitude have occurred in the Southern California area within the last 68 years. A partial list of these earthquakes is included in the following table.

List of Historic Earthquakes

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Kilometers)	Direction to Epicenter
Long Beach	March 10, 1933	6.4	64	SE
Kern County	July 21, 1952	7.5	127	NW
San Fernando	February 9, 1971	6.6	39	N
Whittier Narrows	October 1, 1987	5.9	31	E
Sierra Madre	June 28, 1991	5.8	45	NE
Landers	June 28, 1992	7.3	175	E
Big Bear	June 28, 1992	6.4	143	E
Northridge	January 17, 1994	6.7	21	NW
Hector Mine	October 16, 1999	7.1	206	ENE

The site could be subjected to strong ground shaking in the event of an earthquake. However, this hazard is common in Southern California and the effects of ground shaking can be mitigated by proper engineering design and construction in conformance with current building codes and engineering practices.

Slope Stability

The site is currently developed and the site topography is relatively level. The exception is a 3:1 to 4:1 (horizontal to vertical gradient) landscaped slope along the southern portion of the site. Also, engineer-designed basement walls are present beneath the existing structures. These walls are approximately 60 feet high and are part of the subterranean parking structure at the site. There is also a 15- to 20-foot-high retaining wall along Olympic Boulevard, at the southern site boundary.

The site is included in the City of Los Angeles "Hillside Area" and the County of Los Angeles Landslide Inventory Study Area because of its location on a topographic rise. However, there are no natural slopes or steep graded slopes at or adjacent to the site. The gentle gradient of the landscaped slope along the southern portion of the site, and the engineered retaining wall and basement walls are considered to be grossly stable. This condition precludes both slope stability problems and the potential for lurching (earth movement at right angles to a cliff or steep slope during ground shaking). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Additionally, the site is not located within an area identified as having a potential for seismic slope instability (California Division of Mines and Geology, 1999).

Liquefaction

Liquefaction potential is greatest where the ground water level is shallow, and submerged loose, fine sands occur within a depth of about 15 meters (50 feet) or less. Liquefaction potential decreases as grain size and clay and gravel content increase. As ground acceleration and shaking duration increase during an earthquake, liquefaction potential increases.

According to the California Division of Mines and Geology (1999), the City of Los Angeles Safety Element (1996), and the County of Los Angeles Seismic Safety Element (1990), the site is not within an area identified as having a potential for liquefaction. Ground water was not encountered in our previous borings within 15 meters of the ground surface. Additionally, the Pleistocene age sediments underlying the site are generally medium dense to dense silty sand sand and firm silty clay and clay silts and are not considered prone to liquefaction. Therefore, the potential for liquefaction and the associated ground deformation beneath the site is considered to be low.

Tsunamis, Inundation, Seiches, and Flooding

The site is not in a coastal area. Therefore, tsunamis (seismic sea waves) are not considered a significant hazard at the site.

The site is not located downslope of any large bodies of water that could adversely affect the site in the event of earthquake-induced seiches (wave oscillations in an enclosed or semi-enclosed body of water).

The site is in an area of minimal flooding potential (Zone C) as defined by the Federal Insurance Administration.

Subsidence

The site is within the Beverly Hills Oil Field. The historic withdrawal of fluids (such as petroleum and ground water) has been known to cause ground subsidence. Documented subsidence associated with petroleum and groundwater extraction (and ongoing tectonic processes in the Los Angeles Basin) has occurred within the boundaries of Beverly Hills Oil Field. Between 1955 and 1970, documented subsidence beneath the site was approximately 0.06 meter (0.2 feet) (Hill et. al., 1979). However, this subsidence is regional in nature and there is no evidence that differential settlement or damage to structures has occurred as a result of this phenomenon at the site or in the general area. Therefore, regional subsidence is not anticipated to adversely affect the structures at the site.

6.7 CONCLUSIONS

Based on the available geologic data, active or potentially active faults with the potential for surface

METHANE MITIGATION MAY BE REQUIRED

DRAFT REPORT OF GEOTECHNICAL CONSULTATION PROPOSED HIGH-RISE OFFICE BUILDING AND RETAIL DEVELOPMENT

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7.0 RECOMMENDATIONS

As discussed, the new planned retail structures will be supported on existing columns. If the new column loads do not exceed the loads imposed by the existing structures, no modifications to the foundations are necessary. Accordingly, no foundation recommendations are required for the new planned retail structures.

For the proposed high-rise office building, detailed supplementary settlement analysis will be required prior to finalizing the foundation plan. Differential settlement between new and existing columns and between columns within and outside the zone of influence of the proposed high-rise building will require careful consideration. Furthermore, difficult and unusual construction constraints due to low overhead room and the existing ventilation pits and tunnels and utility trenches at the lowest parking level necessitate individual analysis of each new proposed column. Accordingly, the general foundation, excavation, and shoring recommendations provided in this report should be verified for each column prior to finalizing the foundation plan.

Numerous new columns extending through the existing parking structure will be installed for support of the proposed high-rise office building. The majority of these new columns may be supported on new and enlarged existing spread footings established at the lowest level of the parking structure. In some areas, several spread footings may have to be combined into partial mat foundations to accommodate the design foundation loads. Shored excavations may be required to construct new footings and to enlarge existing footings adjacent to and within ventilation pits and tunnels.

Per the structural engineer's request, we have also developed recommendations for pile foundations for support of selected new columns and/or underpin selected existing spread footings. It may be possible to support new columns adjacent to pits and tunnels on pile foundations to avoid excavations and shoring. Similarly, existing columns adjacent to new proposed columns may be left intact if the new column is supported on pile foundations. However, due to the limited headroom, construction of pile foundations will be very difficult and costly.

The construction constraints may necessitate the use of different foundation systems for different areas. The proposed high-rise building may be supported partially on new spread footings and partially on pile foundations if the anticipated differential settlement between foundation types is within acceptable levels. The cost and design performance requirements will determine the foundation selection for each column.

7.1 FOUNDATIONS

Spread Footings

Bearing Pressure

The existing footings established at least 2 feet into undisturbed natural soils and at least 4 feet below the adjacent basement slab may be surcharged to impose a maximum allowable bearing pressure of 15,000 pounds per square foot.

New, enlarged, and combined spread footings established at least 2 feet into undisturbed natural soils and at least 4 feet below the adjacent basement slab may be designed to impose a net dead-plus-live load pressure of 15,000 pounds per square foot.

The allowable bearing pressure of existing, enlarged, and new footings may be increased by one-third for wind or seismic loads. The recommended bearing value is a net value, and the weight of concrete in the footings can be taken as 50 pounds per cubic foot; the weight of soil backfill can be neglected when determining the downward loads.

Settlement

The expected settlement of each new column and the expected additional settlement of each existing column should be evaluated after the foundations are selected for each new column. The settlement of new columns may necessitate the installation of temporary structural connections between the new columns and the parking level floor slabs. Based on the column load information furnished to us, we estimate that the maximum settlement of new columns supported on spread footings in the manner recommended will be as much as 1½ inches. Over half of the estimated settlement is expected to occur during construction.

Existing columns will also settle due to the consolidation of foundation soils caused by the new adjacent columns. The settlement of existing columns will be highly dependent on the proximity of new adjacent columns, the foundation type of the new adjacent columns, and whether the existing column's load or footing has been modified.

Lateral Resistance

Lateral loads can be resisted by soil friction and by the passive resistance of the soils. A coefficient of friction of 0.5 can be used between the footings and the floor slab and the supporting soils. The passive resistance of natural soils or properly compacted fill can be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per cubic foot. A one-third increase in the passive value can be used for wind or seismic loads. The frictional resistance and the passive resistance of the soils can be combined without reduction in determining the total lateral resistance.

When considering seismic and wind loads, friction between the soils and the basement walls may also be used to resist lateral loads parallel to the basement walls. The resistance may be computed by multiplying the active earth pressure by a coefficient of friction of 0.3.

Drilled Concrete Piles

The downward and upward capacities of 18-, 24-, and 30-inch-diameter drilled piles are presented in Figure 7, Drilled Pile Capacities. A one-third increase may be used for wind or seismic loads. The pile capacities presented are based on the strength of the soils; the compressive and tensile strength of the pile sections should be checked to verify the structural capacity of the piles.

Piles in groups should be spaced at least 2½ diameters on centers. If the piles are so spaced, no reduction in the downward capacities need be considered due to group action.

Portions of ventilation pit and tunnel walls below a 1:1 plane projected downward from the top of adjacent piles should be strengthened to resist surcharge pressures from the piles. Surcharge pressures that may be used for preliminary design are presented in Section 7.4. We should be provided with the final relative locations of basement walls and piles so that we can evaluate surcharge pressures for the final design.

Alternatively, the portion of the piles above a 1:1 plane projecting upward from the base of adjacent basement walls may be isolated from the surrounding soils. The isolation for drilled cast-in-place piles can be achieved by using a Sonotube or equivalent form. Piles with portions isolated from surrounding soils should be lengthened by the isolation length.

Settlement

The total settlement of the columns supported on drilled cast-in-place concrete piles is anticipated to be on the order of ½ inch or less. The settlement of existing adjacent columns due to the settlement of new columns supported on pile foundations is anticipated to be on the order of ¼ inch or less. Over half of the estimated settlement is expected to occur during construction.

Lateral Pile Capacities

Lateral loads may be resisted by the piles, by the passive resistance of the soils against pile caps and grade beams, and by friction between the floor slabs and the supporting soils. The lateral capacity of the piles will depend on the permissible deflection and on the degree of fixity at the top of the pile. The lateral capacities of drilled, cast-in-place concrete piles are presented in the tables below.

Diameter	¼-inch Groundline Deflection					
(inches)	Free Head			Fixed Head		
	Lateral Load (kips)	Maximum Induced Moment (foot-kips)	Depth to Zero Moment (feet)	Lateral Load (kips)	Maximum Induced Moment (foot-kips)	Depth to Zero Moment (feet)
18	30	85	20	70	210	20
24	50	160	25	110	410	25
30	75	260	30	160	700	30

Diameter	½-inch Groundline Deflection					
(inches)	Free Head			Fixed Head		
	Lateral Load (kips)	Maximum Induced Moment (foot-kips)	Depth to Zero Moment (feet)	Lateral Load (kips)	Maximum Induced Moment (foot-kips)	Depth to Zero Moment (feet)
18	40	135	20	90	320	20
24	70	260	25	160	680	25
30	100	420	30	240	1200	30

The capacities presented above are for pile lengths at least 10 feet longer than the depth to zero moment and are based on the strength of the soils; the pile sections should be checked to verify the structural capacity of the piles. The lateral capacity and decrease in the bending moment with depth are based in part on the assumption that any required backfill adjacent to the pile caps and grade beams will be properly compacted.

For piles in groups spaced at least 2½ diameters on centers, no reduction in the lateral capacity need be considered for the first row of piles. For subsequent rows in the direction parallel to loading, piles in groups spaced closer than 7 pile diameters on centers will have a reduction in lateral capacity due to group effects. In the direction parallel to loading, the lateral capacity of piles in groups spaced at 2½ diameters on centers may be assumed to be reduced by half. The reduction of lateral capacity in the direction parallel to loading for other pile spacings may be interpolated between no reduction for piles spaced at 7 pile diameters on centers and the reduction for piles spaced at 2½ diameters on centers.

A coefficient of friction of 0.5 may be used between the floor slabs and the supporting soils. The passive resistance of the natural soils or properly compacted fill against pile caps and grade beams may be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per cubic foot. A one-third increase in the quoted passive value may be used for wind or seismic loads.

The resistance of the piles and the passive resistance of the soils against pile caps and grade beams may be combined without reduction in determining the total lateral resistance.

Laterally loaded piles, pile caps, and grade beams within 7 pile diameters of adjacent ventilation pit and tunnel walls may impose surcharge pressures when loaded in the direction of the walls. Unless pile caps and grade beams will not be designed to resist lateral loads perpendicular to these walls, the capacity of adjacent basement walls to resist surcharge pressures should be verified. We can provide anticipated surcharge pressures once the relative configuration of the basement and pile foundation system is finalized.

Pile Installation

The drilling of the pile shafts and the placing of the concrete should be observed continuously by personnel from our firm to verify that the desired diameter and depth of piles are achieved.

Caving was encountered within the sand deposits in several large diameter borings performed during our previous explorations at the site. Accordingly, caving should be anticipated during the installation of piling. A volume of concrete greater than the computed amount may be required to fill drilled holes. Precautions should be taken during drilling to minimize caving. The speed of the drill rig may have to be reduced to avoid excessive vibration and further caving.

Piles spaced less than 5 diameters on centers shall be drilled and filled alternately, with the concrete permitted to set at least 8 hours before drilling an adjacent hole. Pile excavations should be filled with concrete as soon after drilling and inspection as possible; the holes should not be left open overnight. The concrete should be placed with special equipment so that the concrete is not allowed to fall freely more than 5 feet and to prevent concrete from striking the walls of the excavations.

Groundwater was not encountered in our borings to a depth of about 45 feet below the lowest parking level, however should piles extend below water levels, special consideration and/or precautions may be necessary. These include removal of water from the drilled shaft prior to concrete placement or placement of concrete under water.

Post-grouted Pin Piles

Due to the previously discussed difficult construction constraints, specialty pile foundations may need to be considered to achieve the desired capacities. For example, post-grouted pin piles can be installed within the low headroom of the lowest parking level to depths sufficient to achieve the desired downward and uplift capacities. However, pin piles can not be relied on to provide lateral resistance.

For preliminary design, it may be assumed that 7-inch-diameter post-grouted pin piles with a bonded length of 70 feet may develop an allowable downward and uplift capacity of 400 kips. The actual capacity of each installed pile will need to be verified using load tests.

Piles in groups should be spaced at least 6 diameters on centers. In general, if the piles are so spaced, no reduction in the downward capacities need be considered due to group action. However, each pile cap should be evaluated on case by case basis before finalizing the foundation plan.

Portions of ventilation pit and tunnel walls below a 1:1 plane projected downward from the top of bonded portion of the piles should be strengthened to resist surcharge pressures from the piles. Surcharge pressures that may be used for preliminary design are presented in Section 7.4. We should be provided with the final relative locations of basement walls and piles so that we can evaluate surcharge pressures for the final design.

Alternatively, the portion of the piles above a 1:1 plane projecting upward from the base of adjacent basement walls may be isolated from the surrounding soils. The isolation for pin piles can be achieved by extending the cased length to the desired depth.

7.2 DYNAMIC CHARACTERISTICS

Response Spectra

Ground motions were postulated corresponding to the Design Basis Earthquake (DBE) having a 10% probability of exceedence during a 50-year time period and the Maximum Capable Earthquake (MCE) having a 10% probability of exceedence during a 100-year time period.

Site-specific response spectra for the two levels of shaking specified were determined by a Probabilistic Seismic Hazard Analysis (PSHA) using the computer program FRISKSP, version 3.01b (Blake, 1995). The response spectra were developed using the ground motion attenuation relations for a type "C" site classification discussed in Boore et al. (1993). Dispersion in the Boore et al. ground motion attenuation relationships was considered by inclusion of the standard deviation of the ground motion data in the attenuation relationship used in the PSHA. The response spectra for the DBE and MCE are presented in Figures 5 and 6, respectively, for structural damping values of 2%, 5%, and 10%.

Site Coefficient And Seismic Zonation

The site coefficient, S, can be determined as established in the Earthquake Regulations under Section 1629 of the UBC, 1997 edition, for seismic design of the proposed towers. Based on a review of the local soil and geologic conditions, the site may be classified as Soil Profile Type S_C, as specified in the 1997 code. The site is located within UBC Seismic Zone 4.

The site is near the Santa Monica fault, which has been determined to be a Type B seismic source by the California Division of Mines and Geology. According to Map M-32 in the 1998 publication from the International Conference of Building Officials entitled "Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada," the proposed site is located at a distance of 2.0 kilometers from the Santa Monica fault. At this distance for a seismic source type B, the near source factors, N_a and N_v, are to be taken as 1.3 and 1.6, respectively, based on Tables 16-S and 16-T of the 1997 UBC.

7.3 FLOOR SLAB SUPPORT

If the subgrade is prepared as recommended in Section 7.6, any replacement floor slabs may be supported on grade. Any portion of the existing subdrain system disturbed or damaged during construction should be replaced and repaired prior to installation of the replacement floor slab.

Relatively non-expansive natural sandy soils exposed at the subgrade elevation may be used for floor slab support. Any clay soils exposed in the subgrade elevation for proposed replacement floor slabs should be removed and replaced with relatively non-expansive properly compacted fill to provide a minimum 2-foot-thick layer of relatively non-expansive soils.

Construction activities and exposure to the environment can cause deterioration of the prepared subgrade. Therefore, we recommend our that our field representative observe the condition of the final subgrade soils immediately prior to slab-on-grade construction, and, if necessary, perform further density and moisture content tests to determine the suitability of the final prepared subgrade.

For determining the required thickness of portland cement concrete (PCC) slabs supported on a 2-foot-thick layer of relatively non-expansive soils compacted to at least 90%, a modulus of subgrade reaction of 150 pounds per cubic inch may be used. The modulus of subgrade reaction was estimated from published empirical data. Plate load tests were not performed.

A low-slump concrete should be used to reduce possible curling of the slab. A 2-inch-thick layer of coarse sand can be placed over the vapor retarding membrane to reduce slab curling. If this sand bedding is used, care should be taken during the placement of the concrete to prevent displacement of the sand. The concrete slab should be allowed to cure properly before placing vinyl or other moisture-sensitive floor covering.

7.4 EXCAVATIONS AND SHORING

Excavations

Excavations up to 20 feet deep will be required for the proposed construction of new and enlarged footings within and adjacent to the existing ventilation pits and tunnels. No exceptional difficulties due to the soil conditions are anticipated in excavating at the site. Conventional earthmoving equipment may be used.

Where the necessary space is available, temporary unsurcharged slopes, up to 20 feet high, may be made at 1:1 in lieu of shoring. Unshored excavations should not extend below a plane drawn at 1½:1 (horizontal to vertical) extending downward from adjacent existing footings.

Excavations should be observed by personnel of our firm so that any necessary modifications based on variations in the soil conditions can be made. Traffic or any surcharged loading should be no closer than 10 feet from the tops of the sloped excavations. A greater setback may be necessary when considering heavy vehicles; we should be advised of such heavy vehicle loadings so that specific setback requirements can be established. All applicable safety requirements and regulations, including OSHA regulations, should be met.

Shoring

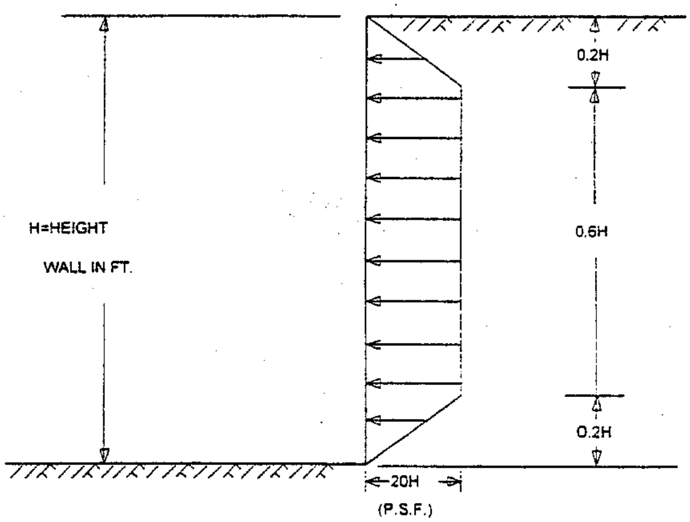
The following general information on shoring design is provided to allow a detailed cost estimation of alternative foundation systems. Installation of spread footings at and adjacent to the ventilation pits and tunnels may require shoring. We can provide additional required data as the design progresses. Also, we suggest that our firm review the final shoring plans and specifications prior to bidding or negotiating with a shoring contractor.

Lateral Pressures

For design of cantilevered shoring, a triangular distribution of lateral earth pressure may be used. It may be assumed that the retained soils with a level surface behind the cantilevered shoring will exert a lateral pressure equal to that developed by a fluid with a density of 25 pounds per cubic foot.

To reduce the potential of excessive deflection of the upper portions of the shoring system, especially adjacent to existing structures and heavily used roadways, braced or tied-back shoring may be used. However, considering the limited headroom and confined construction space, the use of tiebacks will not be practical.

For the design of braced shoring, we recommend the use of a trapezoidal distribution of earth pressure. The recommended pressure distribution, for the case where the grade is level behind the shoring, is illustrated in the diagram below with the maximum pressure equal to 20H in pounds per square foot, where H is the height of the shoring in feet.



In addition to the recommended earth pressure, the upper 10 feet of shoring adjacent automobile traffic should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the shoring due to normal vehicular traffic. The upper 10 feet of shoring adjacent heavy construction traffic should be designed to resist a uniform lateral pressure of 200 pounds per square foot. If the traffic is kept back at least 10 feet from the shoring, the traffic surcharge may be neglected.

Furthermore, the shoring system adjacent to existing footings should also be designed to support the lateral surcharge pressures imposed by the adjacent foundations. Portions below a 1:1 plane projected downward from the edge of adjacent spread footings should be designed to resist a surcharge pressure from those footings. For preliminary design, a uniform lateral surcharge pressure of 1/3 of the footing bearing pressure may be used. We should be provided with the actual design bearing pressures of the existing footings and the final relative locations of the shoring walls and existing footings so that we can provide recommended surcharge pressures for the final design.

In addition, portions of the shoring below a 1:1 plane projected downward from the top of any adjacent drilled cast-in-place piles or the bonded portion of post-grouted pin piles should be designed to resist surcharge pressures from the piles. For preliminary design, the following uniform surcharge pressures may be used for a pile supporting 400 kips of downward loads:

Clear Distance Between Wall and Pile (feet)	Surcharge Pressure (pounds per square feet)
3	2,000
6	1,000
9	500

Surcharge pressures should be projected laterally on both sides of the centerline of each pile by a distance equal to the clear distance to the wall. We should be provided with the final relative locations of basement walls and piles so that we can evaluate surcharge pressures for the final design.

Design of Soldier Piles

For the design of soldier piles spaced at least two diameters on centers, the allowable lateral bearing value (passive value) of the soils below the level of excavation may be assumed to be 600 pounds per square foot per foot of depth at the excavated surface, up to a maximum of 6,000 pounds per square foot. To develop the full lateral value, provisions should be taken to assure firm contact between the soldier piles and the undisturbed soils. The concrete placed in the soldier pile excavations may be a lean-mix concrete. However, the concrete used in that portion of the soldier pile which is below the planned excavated level should be of sufficient strength to adequately transfer the imposed loads to the surrounding soils.

The frictional resistance between the soldier piles and the retained earth may be used in resisting downward loads. The coefficient of friction between the soldier piles and the retained earth may be taken as 0.4. (This value is based on the assumption that uniform full bearing will be developed between the steel soldier beam and the lean-mix concrete and between the lean-mix concrete and the retained earth.). In addition, provided that the portion of the soldier piles below the excavated level is backfilled with structural concrete, the soldier piles below the excavated level may be used to resist downward loads. For resisting the downward loads, the frictional resistance between the concrete soldier piles and the soils below the excavated level may be taken equal to 400 pounds per square foot.

Lagging

Continuous lagging may be required between the soldier piles. We recommend that the exposed soils be observed by personnel of our firm to determine the areas where lagging may be omitted. The unlagged soils should be sprayed with an asphaltic emulsion or equivalent to keep the soils from drying. Depending on the length of exposure, the soils may still dry and crack, posing a hazard for personnel working at the base of the shoring. In such an event, it may be necessary to re-spray the soils or apply wire mesh or chain link fencing to the face of the shoring to prevent chunks of soil from falling.

The soldier piles and bracing should be designed for the full anticipated lateral pressure. However, the pressure on the lagging will be less due to arching in the soils. We recommend that the lagging be designed for the recommended earth pressure but limited to a maximum value of 400 pounds per square foot.

Internal Bracing

Internal struts or raker bracing may be used to internally brace the soldier piles. The strut loads should be determined based on the lateral pressures for restrained conditions presented above. The vertical spacing between struts should be designed to reduce ground movements. All struts should be preloaded to eliminate slack and to reduce ground movement.

If necessary to reduce shoring deflection, a preload of 25 percent of the design load may be used. However, it must be noted that a preload of 25 percent of the design load may induce undue loading on adjacent foundations. This possibility should be analyzed on a case-by-case basis.

Procedures to compensate for the effects of temperature changes on the strut loads should be developed and implemented so that proper strut load levels can be monitored and maintained during construction.

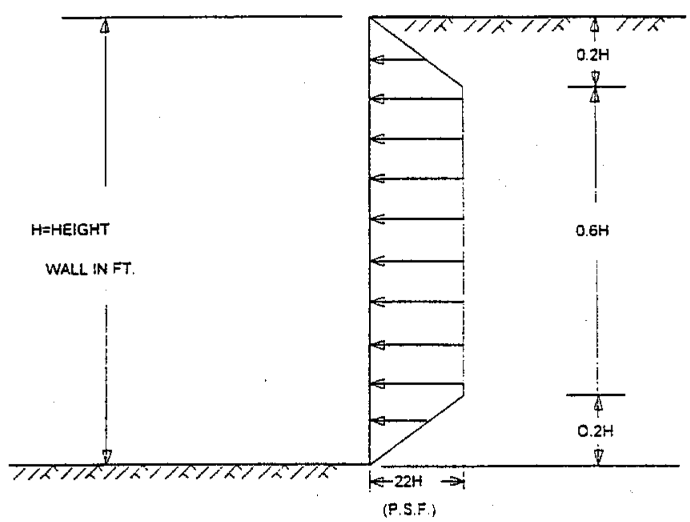
If used, raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent interior footings. For design of such temporary footings, poured with the bearing surface normal to the rakers, which are assumed to be inclined at 45 to 60 degrees with the vertical, a bearing value of 3,000 pounds per square foot may be used, provided the shallowest point of the footing is at least 1 foot below the lowest adjacent grade. To reduce the movement of the shoring, the rakers should be tightly wedged against the footings and/or shoring system.

7.5 RETAINING WALLS

Lateral Pressures

For design of new and/or replacement cantilevered basement walls where the surface of the backfill is level, it may be assumed that the soils will exert a lateral pressure equal to that developed by a fluid with a density of 30 pounds per cubic foot.

For the design of braced basement walls, a trapezoidal distribution of lateral earth pressure plus any surcharge loadings occurring as a result of traffic and foundations of the adjacent buildings should be used. The recommended pressure distribution for the case where the grade is level behind the walls, is illustrated in the following diagram, where the maximum lateral pressure will be 22H in pounds per square foot, where H is the height of the basement wall in feet:



The upper 10 feet of walls adjacent to vehicular traffic areas should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the walls due to normal traffic. If the traffic is kept back at least 10 feet from the walls, the traffic surcharge may be neglected.

Adjacent to existing structures, the basement walls should be designed for the appropriate lateral surcharge pressures as described in Section 7.4, Excavations and Shoring.

Waterproofing

Walls below grade should be waterproofed, or dampproofed, depending on the degree of moisture protection required.

Drainage

Walls below grade should be designed to resist hydrostatic pressures or be provided with a drain pipe or weepholes. The drain could consist of a 4-inch-diameter perforated pipe placed with perforations down at the base of the wall. The pipe should be sloped at least 2 inches in 100 feet and surrounded by filter gravel. The filter gravel should meet the requirements of Class 2 Permeable Material as defined in the current State of California, Department of Transportation, Standard Specifications.

If Class 2 Permeable Material is not available, ¾-inch crushed rock or gravel separated from the on-site soils by an appropriate filter fabric can be used. The crushed rock or gravel should have less than 5% passing a No. 200 sieve.

7.6 GRADING

All required fill should be uniformly well compacted and observed and tested during placement. The on-site soils can be used in any required fill.

Compaction

Any required fill should be placed in loose lifts not more than 8-inches-thick and compacted. The fill should be compacted to at least 90% of the maximum density obtainable by the ASTM Designation D1557-91 method of compaction. The moisture content of the on-site soils at the time of compaction should vary no more than 2% below or above optimum moisture content. The moisture content of the on-site clayey soils at the time of compaction should be between 2% and 4% above optimum moisture content.

Backfill

All required backfill should be mechanically compacted in layers; flooding should not be permitted. Proper compaction of backfill will be necessary to reduce settlement of the backfill and to reduce settlement of overlying slabs and paving. Backfill should be compacted to at least 90% of the maximum dry density obtainable by the ASTM Designation D1557-91 method of compaction. The on-site soils can be used in the compacted backfill.

Material for Fill

The on-site soils, less any debris or organic matter, can be used in required fills. Cobbles larger than 4 inches in diameter should not be used in the fill. Any required import material should consist of relatively non-expansive soils with an expansion index of less than 20. The imported materials should contain sufficient fines (binder material) so as to be relatively impermeable and result in a stable subgrade when compacted. All proposed import materials should be reviewed by our personnel prior to being placed at the site.

7.7 GEOTECHNICAL OBSERVATION

The reworking of the upper soils and the compaction of all required fill should be observed and tested during placement by a representative of our firm. This representative should perform at least the following duties:

- Observe the clearing and grubbing operations for proper removal of all unsuitable materials.
- Observe the exposed subgrade in areas to receive fill and in areas where excavation has resulted in the desired finished subgrade. The representative should also observe proofrolling and delineation of areas requiring overexcavation.
- Evaluate the suitability of on-site and import soils for fill placement; collect and submit soil samples for required or recommended laboratory testing where necessary.
- Observe the fill and backfill for uniformity during placement.
- Test backfill for field density and compaction to determine the percentage of compaction achieved during backfill placement.

- Observe and probe foundation materials to confirm that suitable bearing materials are present at the design foundation depths.

The governmental agencies having jurisdiction over the project should be notified prior to commencement of grading so that the necessary grading permits can be obtained and arrangements can be made for required inspection(s). The contractor should be familiar with the inspection requirements of the reviewing agencies.

8.0 BASIS FOR RECOMMENDATIONS

The recommendations provided in this report are based upon our understanding of the described project information and on our interpretation of the data collected during our and previous subsurface explorations. We have made our recommendations based upon experience with similar subsurface conditions under similar loading conditions. The recommendations apply to the specific project discussed in this report; therefore, any change in the structure configuration, loads, location, or the site grades should be provided to us so that we can review our conclusions and recommendations and make any necessary modifications.

The recommendations provided in this report are also based upon the assumption that the necessary geotechnical observations and testing during construction will be performed by representatives of our firm. The field observation services are considered a continuation of the geotechnical consultation and essential to evaluate that the actual soil conditions are as expected. This also provides for the procedure whereby the client can be advised of unexpected or changed conditions that would require modifications of our original recommendations. In addition, the presence of our representative at the site provides the client with an independent professional opinion regarding the geotechnically related construction procedures. If another firm is retained for the geotechnical observation services, our professional responsibility and liability would be limited to the extent that we would not be the geotechnical engineer of record.

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TABLES

Table 1: Horizontal Ground Motion Pseudo Spectral Velocity in Inches/Second

	2% Damping		5% Damping		10% Damping	
Period in Seconds	DBE 10% in 50 years	MCE 10% in 100 years	DBE 10% in 50 years	MCE 10% in 100 years	DBE 10% in 50 years	MCE 10% in 100 years
0.01	0.34	0.41	0.34	0.41	0.34	0.41
0.03	1.02	1.24	1.02	1.24	1.02	1.24
0.1	7.50	9.26	5.89	7.23	4.91	6.04
0.15	14.74	18.02	11.44	13.90	9.22	11.23
0.2	21.64	26.47	16.59	20.26	13.15	16.13
0.3	32.49	40.13	24.90	30.76	19.44	24.07
0.4	39.70	49.42	30.66	38.34	23.94	29.99
0.5	44.28	55.51	34.61	43.63	27.10	34.21
0.6	47.77	60.21	37.34	47.36	29.20	37.11
0.7	50.20	63.39	39.27	49.94	30.83	39.35
0.8	51.57	65.20	40.39	51.44	31.99	40.93
0.9	52.31	66.09	41.43	52.76	32.80	41.99
1	53.39	67.40	42.08	53.56	33.31	42.61
1.3	56.13	70.94	44.34	56.52	35.32	45.28
1.6	57.56	72.72	45.41	57.80	36.45	46.70
2	58.83	74.29	46.45	59.02	37.17	47.51

9/5/01 TEA
Chkd. JSR 9/6/01

Table 2: Horizontal Ground Motion Pseudo Spectral Acceleration in g's

	2% Damping		5% Damping		10% Damping	
Period in Seconds	DBE 10% in 50 years	SLE 50% in 50 years	DBE 10% in 50 years	SLE 50% in 50 years	DBE 10% in 50 years	SLE 50% in 50 years
0.01	0.55	0.67	0.55	0.67	0.55	0.67
0.03	0.55	0.67	0.55	0.67	0.55	0.67
0.1	1.22	1.51	0.96	1.18	0.80	0.98
0.15	1.60	1.95	1.24	1.51	1.00	1.22
0.2	1.76	2.15	1.35	1.65	1.07	1.31
0.3	1.76	2.18	1.35	1.67	1.05	1.30
0.4	1.61	2.01	1.25	1.56	0.97	1.22
0.5	1.44	1.81	1.13	1.42	0.88	1.11
0.6	1.29	1.63	1.01	1.28	0.79	1.01
0.7	1.17	1.47	0.91	1.16	0.72	0.91
0.8	1.05	1.33	0.82	1.05	0.65	0.83
0.9	0.95	1.19	0.75	0.95	0.59	0.76
1	0.87	1.10	0.68	0.87	0.54	0.69
1.3	0.70	0.89	0.55	0.71	0.44	0.57
1.6	0.58	0.74	0.46	0.59	0.37	0.47
2	0.48	0.60	0.38	0.48	0.30	0.39

9/5/2001 TEA
Chkd. [illegible] 9/6/01

TABLE 3: LIST OF HISTORIC EARTHQUAKES

LIST OF HISTORIC EARTHQUAKES OF MAGNITUDE 4.0 OR
GREATER WITHIN 100 KM OF THE SITE
CAL TECH DATA 1932-2000)

DATE	TIME	LATITUDE	LONGITUDE		Q	DIST	DEPTH	MAGNITUDE
03-11-1933	01:54:07	33.62 N	117.97	W	A	64	.0	6.4
03-11-1933	02:04:00	33.75 N	118.08	W	C	46	.0	4.9
03-11-1933	02:05:00	33.75 N	118.08	W	C	46	.0	4.3
03-11-1933	02:09:00	33.75 N	118.08	W	C	46	.0	5.0
03-11-1933	02:10:00	33.75 N	118.08	W	C	46	.0	4.6
03-11-1933	02:11:00	33.75 N	118.08	W	C	46	.0	4.4
03-11-1933	02:16:00	33.75 N	118.08	W	C	46	.0	4.8
03-11-1933	02:17:00	33.60 N	118.00	W	E	63	.0	4.5
03-11-1933	02:22:00	33.75 N	118.08	W	C	46	.0	4.0
03-11-1933	02:27:00	33.75 N	118.08	W	C	46	.0	4.6
03-11-1933	02:30:00	33.75 N	118.08	W	C	46	.0	5.1
03-11-1933	02:31:00	33.60 N	118.00	W	E	63	.0	4.4
03-11-1933	02:52:00	33.75 N	118.08	W	C	46	.0	4.0
03-11-1933	02:57:00	33.75 N	118.08	W	C	46	.0	4.2
03-11-1933	02:58:00	33.75 N	118.08	W	C	46	.0	4.0
03-11-1933	02:59:00	33.75 N	118.08	W	C	46	.0	4.6
03-11-1933	03:05:00	33.75 N	118.08	W	C	46	.0	4.2
03-11-1933	03:09:00	33.75 N	118.08	W	C	46	.0	4.4
03-11-1933	03:11:00	33.75 N	118.08	W	C	46	.0	4.2
03-11-1933	03:23:00	33.75 N	118.08	W	C	46	.0	5.0
03-11-1933	03:36:00	33.75 N	118.08	W	C	46	.0	4.0
03-11-1933	03:39:00	33.75 N	118.08	W	C	46	.0	4.0
03-11-1933	03:47:00	33.75 N	118.08	W	C	46	.0	4.1
03-11-1933	04:36:00	33.75 N	118.08	W	C	46	.0	4.6
03-11-1933	04:39:00	33.75 N	118.08	W	C	46	.0	4.9
03-11-1933	04:40:00	33.75 N	118.08	W	C	46	.0	4.7
03-11-1933	05:10:22	33.70 N	118.07	W	C	51	.0	5.1
03-11-1933	05:13:00	33.75 N	118.08	W	C	46	.0	4.7
03-11-1933	05:15:00	33.75 N	118.08	W	C	46	.0	4.0
03-11-1933	05:18:04	33.58 N	117.98	W	C	67	.0	5.2
03-11-1933	05:21:00	33.75 N	118.08	W	C	46	.0	4.4
03-11-1933	05:24:00	33.75 N	118.08	W	C	46	.0	4.2
03-11-1933	05:53:00	33.75 N	118.08	W	C	46	.0	4.0
03-11-1933	05:55:00	33.75 N	118.08	W	C	46	.0	4.0
03-11-1933	06:11:00	33.75 N	118.08	W	C	46	.0	4.4

NOTE: Q IS A FACTOR RELATING THE QUALITY OF EPICENTRAL DETERMINATION

A = +- 1 km horizontal distance; +- 2 km depth
B = +- 2 km horizontal distance; +- 5 km depth
C = +- 5 km horizontal distance; no depth restriction
D = >+- 5 km horizontal distance

Event qualities are highly suspect prior to 1990. Many of these event qualities are based on incomplete information according to Caltech.

DATE	TIME	LATITUDE	LONGITUDE		Q	DIST	DEPTH	MAGNITUDE
03-11-1933	06:18:00	33.75 N	118.08	W	C	46	.0	4.2
03-11-1933	06:29:00	33.85 N	118.27	W	C	27	.0	4.4
03-11-1933	06:35:00	33.75 N	118.08	W	C	46	.0	4.2
03-11-1933	06:58:03	33.68 N	118.05	W	C	53	.0	5.5
03-11-1933	07:51:00	33.75 N	118.08	W	C	46	.0	4.2
03-11-1933	07:59:00	33.75 N	118.08	W	C	46	.0	4.1
03-11-1933	08:08:00	33.75 N	118.08	W	C	46	.0	4.5
03-11-1933	08:32:00	33.75 N	118.08	W	C	46	.0	4.2
03-11-1933	08:37:00	33.75 N	118.08	W	C	46	.0	4.0
03-11-1933	08:54:57	33.70 N	118.07	W	C	51	.0	5.1
03-11-1933	09:10:00	33.75 N	118.08	W	C	46	.0	5.1
03-11-1933	09:11:00	33.75 N	118.08	W	C	46	.0	4.4
03-11-1933	09:26:00	33.75 N	118.08	W	C	46	.0	4.1
03-11-1933	10:25:00	33.75 N	118.08	W	C	46	.0	4.0
03-11-1933	10:45:00	33.75 N	118.08	W	C	46	.0	4.0
03-11-1933	11:00:00	33.75 N	118.08	W	C	46	.0	4.0
03-11-1933	11:04:00	33.75 N	118.13	W	C	43	.0	4.6
03-11-1933	11:29:00	33.75 N	118.08	W	C	46	.0	4.0
03-11-1933	11:38:00	33.75 N	118.08	W	C	46	.0	4.0
03-11-1933	11:41:00	33.75 N	118.08	W	C	46	.0	4.2
03-11-1933	11:47:00	33.75 N	118.08	W	C	46	.0	4.4
03-11-1933	12:50:00	33.68 N	118.05	W	C	53	.0	4.4
03-11-1933	13:50:00	33.73 N	118.10	W	C	46	.0	4.4
03-11-1933	13:57:00	33.75 N	118.08	W	C	46	.0	4.0
03-11-1933	14:25:00	33.85 N	118.27	W	C	27	.0	5.0
03-11-1933	14:47:00	33.73 N	118.10	W	C	46	.0	4.4
03-11-1933	14:57:00	33.88 N	118.32	W	C	21	.0	4.9
03-11-1933	15:09:00	33.73 N	118.10	W	C	46	.0	4.4
03-11-1933	15:47:00	33.75 N	118.08	W	C	46	.0	4.0
03-11-1933	16:53:00	33.75 N	118.08	W	C	46	.0	4.8
03-11-1933	19:44:00	33.75 N	118.08	W	C	46	.0	4.0
03-11-1933	19:56:00	33.75 N	118.08	W	C	46	.0	4.2
03-11-1933	22:00:00	33.75 N	118.08	W	C	46	.0	4.4
03-11-1933	22:31:00	33.75 N	118.08	W	C	46	.0	4.4
03-11-1933	22:32:00	33.75 N	118.08	W	C	46	.0	4.1
03-11-1933	22:40:00	33.75 N	118.08	W	C	46	.0	4.4
03-11-1933	23:05:00	33.75 N	118.08	W	C	46	.0	4.2
03-12-1933	00:27:00	33.75 N	118.08	W	C	46	.0	4.4
03-12-1933	00:34:00	33.75 N	118.08	W	C	46	.0	4.0
03-12-1933	04:48:00	33.75 N	118.08	W	C	46	.0	4.0
03-12-1933	05:46:00	33.75 N	118.08	W	C	46	.0	4.4
03-12-1933	06:01:00	33.75 N	118.08	W	C	46	.0	4.2
03-12-1933	06:16:00	33.75 N	118.08	W	C	46	.0	4.6
03-12-1933	07:40:00	33.75 N	118.08	W	C	46	.0	4.2
03-12-1933	08:35:00	33.75 N	118.08	W	C	46	.0	4.2

DATE	TIME	LATITUDE	LONGITUDE		Q	DIST	DEPTH	MAGNITUDE
03-12-1933	15:02:00	33.75 N	118.08	W	C	46	.0	4.2
03-12-1933	16:51:00	33.75 N	118.08	W	C	46	.0	4.0
03-12-1933	17:38:00	33.75 N	118.08	W	C	46	.0	4.5
03-12-1933	18:25:00	33.75 N	118.08	W	C	46	.0	4.1
03-12-1933	21:28:00	33.75 N	118.08	W	C	46	.0	4.1
03-12-1933	23:54:00	33.75 N	118.08	W	C	46	.0	4.5
03-13-1933	03:43:00	33.75 N	118.08	W	C	46	.0	4.1
03-13-1933	04:32:00	33.75 N	118.08	W	C	46	.0	4.7
03-13-1933	06:17:00	33.75 N	118.08	W	C	46	.0	4.0
03-13-1933	13:18:28	33.75 N	118.08	W	C	46	.0	5.3
03-13-1933	15:32:00	33.75 N	118.08	W	C	46	.0	4.1
03-13-1933	19:29:00	33.75 N	118.08	W	C	46	.0	4.2
03-14-1933	00:36:00	33.75 N	118.08	W	C	46	.0	4.2
03-14-1933	12:19:00	33.75 N	118.08	W	C	46	.0	4.5
03-14-1933	19:01:50	33.62 N	118.02	W	C	61	.0	5.1
03-14-1933	22:42:00	33.75 N	118.08	W	C	46	.0	4.1
03-15-1933	02:08:00	33.75 N	118.08	W	C	46	.0	4.1
03-15-1933	04:32:00	33.75 N	118.08	W	C	46	.0	4.1
03-15-1933	05:40:00	33.75 N	118.08	W	C	46	.0	4.2
03-15-1933	11:13:32	33.62 N	118.02	W	C	61	.0	4.9
03-16-1933	14:56:00	33.75 N	118.08	W	C	46	.0	4.0
03-16-1933	15:29:00	33.75 N	118.08	W	C	46	.0	4.2
03-16-1933	15:30:00	33.75 N	118.08	W	C	46	.0	4.1
03-17-1933	16:51:00	33.75 N	118.08	W	C	46	.0	4.1
03-18-1933	20:52:00	33.75 N	118.08	W	C	46	.0	4.2
03-19-1933	21:23:00	33.75 N	118.08	W	C	46	.0	4.2
03-20-1933	13:58:00	33.75 N	118.08	W	C	46	.0	4.1
03-21-1933	03:26:00	33.75 N	118.08	W	C	46	.0	4.1
03-23-1933	08:40:00	33.75 N	118.08	W	C	46	.0	4.1
03-23-1933	18:31:00	33.75 N	118.08	W	C	46	.0	4.1
03-25-1933	13:46:00	33.75 N	118.08	W	C	46	.0	4.1
03-30-1933	12:25:00	33.75 N	118.08	W	C	46	.0	4.4
03-31-1933	10:49:00	33.75 N	118.08	W	C	46	.0	4.1
04-01-1933	06:42:00	33.75 N	118.08	W	C	46	.0	4.2
04-02-1933	08:00:00	33.75 N	118.08	W	C	46	.0	4.0
04-02-1933	15:36:00	33.75 N	118.08	W	C	46	.0	4.0
05-16-1933	20:58:55	33.75 N	118.17	W	C	41	.0	4.0
08-04-1933	04:17:48	33.75 N	118.18	W	C	40	.0	4.0
10-02-1933	09:10:17	33.78 N	118.13	W	A	40	.0	5.4
10-02-1933	13:26:01	33.62 N	118.02	W	C	61	.0	4.0
10-25-1933	07:00:46	33.95 N	118.13	W	C	28	.0	4.3
11-13-1933	21:28:00	33.87 N	118.20	W	C	29	.0	4.0
11-20-1933	10:32:00	33.78 N	118.13	W	B	40	.0	4.0
01-09-1934	14:10:00	34.10 N	117.68	W	A	68	.0	4.5
01-18-1934	02:14:00	34.10 N	117.68	W	A	68	.0	4.0

DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
01-20-1934	21:17:00	33.62 N	118.12 W	B	56	.0	4.5
04-17-1934	18:33:00	33.57 N	117.98 W	C	67	.0	4.0
10-17-1934	09:38:00	33.63 N	118.40 W	B	47	.0	4.0
11-16-1934	21:26:00	33.75 N	118.00 W	B	51	.0	4.0
06-11-1935	18:10:00	34.72 N	118.97 W	B	89	.0	4.0
06-19-1935	11:17:00	33.72 N	117.52 W	B	91	.0	4.0
07-13-1935	10:54:16	34.20 N	117.90 W	A	50	.0	4.7
12-25-1935	17:15:00	33.60 N	118.02 W	B	63	.0	4.5
02-23-1936	22:20:42	34.13 N	117.34 W	A	100	10.0	4.5
02-26-1936	09:33:27	34.14 N	117.34 W	A	100	10.0	4.0
08-22-1936	05:21:00	33.77 N	117.82 W	B	64	.0	4.0
10-29-1936	22:35:36	34.38 N	118.62 W	C	41	10.0	4.0
01-15-1937	18:35:47	33.56 N	118.06 W	B	64	10.0	4.0
03-19-1937	01:23:38	34.11 N	117.43 W	A	91	10.0	4.0
07-07-1937	11:12:00	33.57 N	117.98 W	B	67	.0	4.0
09-01-1937	13:48:08	34.21 N	117.53 W	A	83	10.0	4.5
09-01-1937	16:35:33	34.18 N	117.55 W	A	81	10.0	4.5
05-21-1938	09:44:00	33.62 N	118.03 W	B	60	.0	4.0
05-31-1938	08:34:55	33.70 N	117.51 W	B	92	10.0	5.2
07-05-1938	18:06:55	33.68 N	117.55 W	A	90	10.0	4.5
08-06-1938	22:00:55	33.72 N	117.51 W	B	92	10.0	4.0
08-31-1938	03:18:14	33.76 N	118.25 W	A	36	10.0	4.5
11-29-1938	19:21:15	33.90 N	118.43 W	A	17	10.0	4.0
12-07-1938	03:38:00	34.00 N	118.42 W	B	6	.0	4.0
12-27-1938	10:09:28	34.13 N	117.52 W	B	83	10.0	4.0
11-04-1939	21:41:00	33.77 N	118.12 W	B	42	.0	4.0
12-27-1939	19:28:49	33.78 N	118.20 W	A	36	.0	4.7
01-13-1940	07:49:07	33.78 N	118.13 W	B	40	.0	4.0
02-08-1940	16:56:17	33.70 N	118.07 W	B	51	.0	4.0
02-11-1940	19:24:10	33.98 N	118.30 W	B	13	.0	4.0
04-18-1940	18:43:43	34.03 N	117.35 W	A	98	.0	4.4
05-18-1940	09:15:12	34.60 N	118.90 W	C	75	.0	4.0
06-05-1940	08:27:27	33.83 N	117.40 W	B	97	.0	4.0
07-20-1940	04:01:13	33.70 N	118.07 W	B	51	.0	4.0
10-11-1940	05:57:12	33.77 N	118.45 W	A	32	.0	4.7
10-12-1940	00:24:00	33.78 N	118.42 W	B	30	.0	4.0
10-14-1940	20:51:11	33.78 N	118.42 W	B	30	.0	4.0
11-01-1940	07:25:03	33.78 N	118.42 W	B	30	.0	4.0
11-01-1940	20:00:46	33.63 N	118.20 W	B	51	.0	4.0
11-02-1940	02:58:26	33.78 N	118.42 W	B	30	.0	4.0
01-30-1941	01:34:46	33.97 N	118.05 W	A	35	.0	4.1
03-22-1941	08:22:40	33.52 N	118.10 W	B	67	.0	4.0
03-25-1941	23:43:41	34.22 N	117.47 W	B	89	.0	4.0
04-11-1941	01:20:24	33.95 N	117.58 W	B	78	.0	4.0
10-22-1941	06:57:18	33.82 N	118.22 W	A	32	.0	4.8

DATE	TIME	LATITUDE	LONGITUDE		Q	DIST	DEPTH	MAGNITUDE
11-14-1941	08:41:36	33.78 N	118.25	W	A	34	.0	4.8
04-16-1942	07:28:33	33.37 N	118.15	W	C	80	.0	4.0
09-03-1942	14:06:01	34.48 N	118.98	W	C	71	.0	4.5
09-04-1942	06:34:33	34.48 N	118.98	W	C	71	.0	4.5
04-06-1943	22:36:24	34.68 N	119.00	W	C	88	.0	4.0
10-24-1943	00:29:21	33.93 N	117.37	W	C	98	.0	4.0
06-19-1944	00:03:33	33.87 N	118.22	W	B	28	.0	4.5
06-19-1944	03:06:07	33.87 N	118.22	W	C	28	.0	4.4
02-24-1946	06:07:52	34.40 N	117.80	W	C	68	.0	4.1
06-01-1946	11:06:31	34.42 N	118.83	W	C	56	.0	4.1
03-01-1948	08:12:13	34.17 N	117.53	W	B	82	.0	4.7
04-16-1948	22:26:24	34.02 N	118.97	W	B	51	.0	4.7
10-03-1948	02:46:28	34.18 N	117.58	W	A	78	.0	4.0
01-11-1950	21:41:35	33.94 N	118.20	W	A	23	.4	4.1
01-24-1950	21:56:59	34.67 N	118.83	W	C	78	.0	4.0
02-26-1950	00:06:22	34.62 N	119.08	W	C	88	.0	4.7
08-22-1950	22:47:58	34.15 N	119.35	W	B	87	.0	4.2
09-22-1951	08:22:39	34.12 N	117.34	W	A	99	11.9	4.3
02-10-1952	13:50:55	33.58 N	119.18	W	C	88	.0	4.0
07-22-1952	07:44:55	34.87 N	118.87	W	A	99	.0	4.1
08-23-1952	10:09:07	34.52 N	118.20	W	A	55	13.1	5.1
10-26-1954	16:22:26	33.73 N	117.47	W	B	95	.0	4.1
11-17-1954	23:03:51	34.50 N	119.12	W	B	81	.0	4.4
05-15-1955	17:03:25	34.12 N	117.48	W	A	87	7.6	4.0
05-29-1955	16:43:35	33.99 N	119.06	W	B	60	17.4	4.1
01-03-1956	00:25:48	33.72 N	117.50	W	B	92	13.7	4.7
02-07-1956	02:16:56	34.53 N	118.64	W	B	57	16.0	4.2
02-07-1956	03:16:38	34.59 N	118.61	W	A	62	2.6	4.6
03-25-1956	03:32:02	33.60 N	119.11	W	A	81	8.2	4.2
03-18-1957	18:56:28	34.12 N	119.22	W	B	75	13.8	4.7
06-28-1960	20:00:48	34.12 N	117.47	W	A	87	12.0	4.1
10-04-1961	02:21:31	33.85 N	117.75	W	B	65	4.3	4.1
10-20-1961	19:49:50	33.65 N	117.99	W	B	59	4.6	4.3
10-20-1961	20:07:14	33.66 N	117.98	W	B	60	6.1	4.0
10-20-1961	21:42:40	33.67 N	117.98	W	B	59	7.2	4.0
10-20-1961	22:35:34	33.67 N	118.01	W	B	57	5.6	4.1
11-20-1961	08:53:34	33.68 N	117.99	W	B	57	4.4	4.0
09-14-1963	03:51:16	33.54 N	118.34	W	B	57	2.2	4.2
08-30-1964	22:57:37	34.27 N	118.44	W	B	24	15.4	4.0
01-01-1965	08:04:18	34.14 N	117.52	W	B	83	5.9	4.4
04-15-1965	20:08:33	34.13 N	117.43	W	B	92	5.5	4.5
07-16-1965	07:46:22	34.49 N	118.52	W	B	49	15.1	4.0
01-08-1967	07:37:30	33.63 N	118.47	W	B	47	11.4	4.0
01-08-1967	07:38:05	33.66 N	118.41	W	C	44	17.7	4.0
06-15-1967	04:58:05	34.00 N	117.97	W	B	41	10.0	4.1

DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
02-28-1969	04:56:12	34.57 N	118.11 W	A	63	5.3	4.3
05-05-1969	16:02:09	34.30 N	117.57 W	B	83	8.8	4.4
10-27-1969	13:16:02	33.55 N	117.81 W	B	80	6.5	4.5
10-31-1969	10:39:28	33.43 N	119.10 W	B	94	7.3	4.7
09-12-1970	14:10:11	34.27 N	117.52 W	A	86	8.0	4.1
09-12-1970	14:30:52	34.27 N	117.54 W	A	84	8.0	5.2
09-13-1970	04:47:48	34.28 N	117.55 W	A	83	8.0	4.4
02-09-1971	14:00:41	34.41 N	118.40 W	B	39	8.4	6.6
02-09-1971	14:01:08	34.41 N	118.40 W	D	39	8.0	5.8
02-09-1971	14:01:33	34.41 N	118.40 W	D	39	8.0	4.2
02-09-1971	14:01:40	34.41 N	118.40 W	D	39	8.0	4.1
02-09-1971	14:01:50	34.41 N	118.40 W	D	39	8.0	4.5
02-09-1971	14:01:54	34.41 N	118.40 W	D	39	8.0	4.2
02-09-1971	14:01:59	34.41 N	118.40 W	D	39	8.0	4.1
02-09-1971	14:02:03	34.41 N	118.40 W	D	39	8.0	4.1
02-09-1971	14:02:30	34.41 N	118.40 W	D	39	8.0	4.3
02-09-1971	14:02:31	34.41 N	118.40 W	D	39	8.0	4.7
02-09-1971	14:02:44	34.41 N	118.40 W	D	39	8.0	5.8
02-09-1971	14:03:25	34.41 N	118.40 W	D	39	8.0	4.4
02-09-1971	14:03:46	34.41 N	118.40 W	D	39	8.0	4.1
02-09-1971	14:04:07	34.41 N	118.40 W	D	39	8.0	4.1
02-09-1971	14:04:34	34.41 N	118.40 W	C	39	8.0	4.2
02-09-1971	14:04:39	34.41 N	118.40 W	D	39	8.0	4.1
02-09-1971	14:04:44	34.41 N	118.40 W	D	39	8.0	4.1
02-09-1971	14:04:46	34.41 N	118.40 W	D	39	8.0	4.2
02-09-1971	14:05:41	34.41 N	118.40 W	D	39	8.0	4.1
02-09-1971	14:05:50	34.41 N	118.40 W	D	39	8.0	4.1
02-09-1971	14:07:10	34.41 N	118.40 W	D	39	8.0	4.0
02-09-1971	14:07:30	34.41 N	118.40 W	D	39	8.0	4.0
02-09-1971	14:07:45	34.41 N	118.40 W	D	39	8.0	4.5
02-09-1971	14:08:04	34.41 N	118.40 W	D	39	8.0	4.0
02-09-1971	14:08:07	34.41 N	118.40 W	D	39	8.0	4.2
02-09-1971	14:08:38	34.41 N	118.40 W	D	39	8.0	4.5
02-09-1971	14:08:53	34.41 N	118.40 W	D	39	8.0	4.6
02-09-1971	14:10:21	34.36 N	118.31 W	B	35	5.0	4.7
02-09-1971	14:10:28	34.41 N	118.40 W	D	39	8.0	5.3
02-09-1971	14:16:12	34.34 N	118.33 W	C	32	11.1	4.1
02-09-1971	14:19:50	34.36 N	118.41 W	B	33	11.8	4.0
02-09-1971	14:34:36	34.34 N	118.64 W	C	38	-2.0	4.9
02-09-1971	14:39:17	34.39 N	118.36 W	C	37	-1.6	4.0
02-09-1971	14:40:17	34.43 N	118.40 W	C	42	-2.0	4.1
02-09-1971	14:43:46	34.31 N	118.45 W	B	28	6.2	5.2
02-09-1971	15:58:20	34.33 N	118.33 W	B	32	14.2	4.8
02-09-1971	16:19:26	34.46 N	118.43 W	B	44	-1.0	4.2
02-10-1971	03:12:12	34.37 N	118.30 W	B	36	.8	4.0

DATE	TIME	LATITUDE	LONGITUDE		Q	DIST	DEPTH	MAGNITUDE
02-10-1971	05:06:36	34.41 N	118.33	W	A	40	4.7	4.3
02-10-1971	05:18:07	34.43 N	118.41	W	A	41	5.8	4.5
02-10-1971	11:31:34	34.38 N	118.46	W	A	37	6.0	4.2
02-10-1971	13:49:53	34.40 N	118.42	W	A	38	9.7	4.3
02-10-1971	14:35:26	34.36 N	118.49	W	A	34	4.4	4.2
02-10-1971	17:38:55	34.40 N	118.37	W	A	38	6.2	4.2
02-10-1971	18:54:41	34.45 N	118.44	W	A	43	8.1	4.2
02-21-1971	05:50:52	34.40 N	118.44	W	A	38	6.9	4.7
02-21-1971	07:15:11	34.39 N	118.43	W	A	37	7.2	4.5
03-07-1971	01:33:40	34.35 N	118.46	W	A	33	3.3	4.5
03-25-1971	22:54:09	34.36 N	118.47	W	A	34	4.6	4.2
03-30-1971	08:54:43	34.30 N	118.46	W	A	27	2.6	4.1
03-31-1971	14:52:22	34.29 N	118.51	W	A	27	2.1	4.6
04-01-1971	15:03:03	34.43 N	118.41	W	A	41	8.0	4.1
04-02-1971	05:40:25	34.28 N	118.53	W	A	27	3.0	4.0
04-15-1971	11:14:32	34.26 N	118.58	W	B	28	4.2	4.2
04-25-1971	14:48:06	34.37 N	118.31	W	B	36	-2.0	4.0
06-21-1971	16:01:08	34.27 N	118.53	W	B	26	4.1	4.0
06-22-1971	10:41:19	33.75 N	117.48	W	B	93	8.0	4.2
07-27-1972	00:31:17	34.78 N	118.90	W	A	92	8.0	4.4
02-21-1973	14:45:57	34.06 N	119.04	W	B	57	8.0	5.3
08-06-1973	23:29:16	33.99 N	119.48	W	A	98	16.9	5.0
03-09-1974	00:54:31	34.40 N	118.47	W	C	38	24.4	4.7
08-14-1974	14:45:55	34.43 N	118.37	W	A	42	8.2	4.2
01-01-1976	17:20:12	33.97 N	117.89	W	A	50	6.2	4.2
04-08-1976	15:21:38	34.35 N	118.66	W	A	39	14.5	4.6
08-12-1977	02:19:26	34.38 N	118.46	W	B	36	9.5	4.5
09-24-1977	21:28:24	34.46 N	118.41	W	C	45	5.0	4.2
05-23-1978	09:16:50	33.91 N	119.17	W	C	71	6.0	4.0
01-01-1979	23:14:38	33.94 N	118.68	W	B	28	11.3	5.2
10-17-1979	20:52:37	33.93 N	118.67	W	C	27	5.5	4.2
10-19-1979	12:22:37	34.21 N	117.53	W	B	83	4.9	4.1
09-04-1981	15:50:50	33.65 N	119.09	W	C	77	6.0	5.5
10-23-1981	17:28:17	33.64 N	119.01	W	C	72	6.0	4.6
10-23-1981	19:15:52	33.62 N	119.02	W	A	74	14.8	4.6
04-13-1982	11:02:12	34.06 N	118.97	W	A	51	12.1	4.0
05-25-1982	13:44:30	33.55 N	118.21	W	A	60	12.6	4.3
01-08-1983	07:19:30	34.13 N	117.45	W	A	89	7.8	4.1
02-27-1984	10:18:15	33.47 N	118.06	W	C	73	6.0	4.0
06-12-1984	00:27:52	34.54 N	118.99	W	A	76	11.7	4.1
10-26-1984	17:20:43	34.02 N	118.99	W	A	53	13.3	4.6
04-03-1985	04:04:50	34.38 N	119.04	W	A	68	24.9	4.0
02-21-1987	23:15:29	34.13 N	117.45	W	A	90	8.5	4.0
10-01-1987	14:42:20	34.06 N	118.08	W	A	31	9.5	5.9
10-01-1987	14:45:41	34.05 N	118.10	W	A	29	13.6	4.7

DATE	TIME	LATITUDE	LONGITUDE		Q	DIST	DEPTH	MAGNITUDE
10-01-1987	14:48:03	34.08 N	118.09	W	A	30	11.7	4.1
10-01-1987	14:49:05	34.06 N	118.10	W	A	29	11.7	4.7
10-01-1987	15:12:31	34.05 N	118.09	W	A	30	10.8	4.7
10-01-1987	15:59:53	34.05 N	118.09	W	A	30	10.4	4.0
10-04-1987	10:59:38	34.07 N	118.10	W	A	29	8.3	5.3
10-24-1987	23:58:33	33.68 N	119.06	W	A	73	12.2	4.1
02-11-1988	15:25:55	34.08 N	118.05	W	A	34	12.5	4.7
06-26-1988	15:04:58	34.14 N	117.71	W	A	66	7.9	4.7
11-20-1988	05:39:28	33.51 N	118.07	W	C	69	6.0	4.9
12-03-1988	11:38:26	34.15 N	118.13	W	A	28	14.3	5.0
01-19-1989	06:53:28	33.92 N	118.63	W	A	25	11.9	5.0
02-18-1989	07:17:04	34.01 N	117.74	W	A	63	3.3	4.1
04-07-1989	20:07:30	33.62 N	117.90	W	A	68	12.9	4.7
06-12-1989	16:57:18	34.03 N	118.18	W	A	22	15.6	4.6
06-12-1989	17:22:25	34.02 N	118.18	W	A	22	15.5	4.4
12-28-1989	09:41:08	34.19 N	117.39	W	A	96	14.6	4.3
02-28-1990	23:43:36	34.14 N	117.70	W	A	67	4.5	5.4
03-01-1990	00:34:57	34.13 N	117.70	W	A	66	4.4	4.0
03-01-1990	03:23:03	34.15 N	117.72	W	A	65	11.4	4.7
03-02-1990	17:26:25	34.15 N	117.69	W	A	67	5.6	4.7
04-17-1990	22:32:27	34.11 N	117.72	W	A	64	3.6	4.8
06-28-1991	14:43:54	34.27 N	117.99	W	A	45	9.1	5.8
06-28-1991	17:00:55	34.25 N	117.99	W	A	45	9.5	4.3
07-05-1991	17:41:57	34.50 N	118.56	W	A	51	10.9	4.1
01-17-1994	12:30:55	34.21 N	118.54	W	A	21	18.4	6.7
01-17-1994	12:30:55	34.22 N	118.54	W	A	21	17.4	6.6
01-17-1994	12:31:58	34.27 N	118.49	W	C	25	6.0	5.9
01-17-1994	12:34:18	34.31 N	118.47	W	C	28	6.0	4.4
01-17-1994	12:39:39	34.26 N	118.54	W	C	26	6.0	4.9
01-17-1994	12:40:09	34.32 N	118.51	W	C	30	6.0	4.8
01-17-1994	12:40:36	34.34 N	118.61	W	C	36	6.0	5.2
01-17-1994	12:54:33	34.31 N	118.46	W	C	28	6.0	4.0
01-17-1994	12:55:46	34.28 N	118.58	W	C	29	6.0	4.1
01-17-1994	13:06:28	34.25 N	118.55	W	C	25	6.0	4.6
01-17-1994	13:26:45	34.32 N	118.46	W	C	29	6.0	4.7
01-17-1994	13:28:13	34.27 N	118.58	W	C	28	6.0	4.0
01-17-1994	13:56:02	34.29 N	118.62	W	C	32	6.0	4.4
01-17-1994	14:14:30	34.33 N	118.44	W	C	31	6.0	4.5
01-17-1994	15:07:03	34.30 N	118.47	W	A	28	2.6	4.2
01-17-1994	15:07:35	34.31 N	118.47	W	A	28	1.6	4.1
01-17-1994	15:54:10	34.38 N	118.63	W	A	40	13.0	4.8
01-17-1994	17:56:08	34.23 N	118.57	W	A	24	19.2	4.6
01-17-1994	19:35:34	34.31 N	118.46	W	A	28	2.3	4.0
01-17-1994	19:43:53	34.37 N	118.64	W	A	40	13.9	4.1
01-17-1994	20:46:02	34.30 N	118.57	W	C	31	6.0	4.9

DATE	TIME	LATITUDE	LONGITUDE		Q	DIST	DEPTH	MAGNITUDE
01-17-1994	22:31.53	34.34 N	118.44	W	C	31	6.0	4.1
01-17-1994	23:33:30	34.33 N	118.70	W	A	40	9.8	5.6
01-17-1994	23:49:25	34.34 N	118.67	W	A	39	8.4	4.0
01-18-1994	00:39:35	34.38 N	118.56	W	A	38	7.2	4.4
01-18-1994	00:40:04	34.39 N	118.54	W	A	39	.0	4.2
01-18-1994	00:43:08	34.38 N	118.70	W	A	44	11.3	5.2
01-18-1994	04:01:26	34.36 N	118.62	W	A	39	.9	4.3
01-18-1994	07:23:56	34.33 N	118.62	W	A	36	14.8	4.0
01-18-1994	11:35:09	34.22 N	118.61	W	A	25	12.1	4.2
01-18-1994	13:24:44	34.32 N	118.56	W	A	32	1.7	4.3
01-18-1994	15:23:46	34.38 N	118.56	W	A	38	7.7	4.8
01-19-1994	04:40:48	34.36 N	118.57	W	A	37	2.6	4.3
01-19-1994	04:43:14	34.37 N	118.71	W	C	44	6.0	4.0
01-19-1994	09:13:10	34.30 N	118.74	W	A	41	13.0	4.1
01-19-1994	14:09:14	34.22 N	118.51	W	A	20	17.5	4.5
01-19-1994	21:09:28	34.38 N	118.71	W	A	45	14.4	5.1
01-19-1994	21:11:44	34.38 N	118.62	W	A	40	11.4	5.1
01-21-1994	18:39:15	34.30 N	118.47	W	A	28	10.6	4.5
01-21-1994	18:39:47	34.30 N	118.48	W	A	27	11.9	4.0
01-21-1994	18:42:28	34.31 N	118.47	W	A	29	7.9	4.2
01-21-1994	18:52:44	34.30 N	118.45	W	A	27	7.6	4.3
01-21-1994	18:53:44	34.30 N	118.46	W	A	27	7.7	4.3
01-23-1994	08:55:08	34.30 N	118.43	W	A	27	6.0	4.1
01-24-1994	04:15:18	34.35 N	118.55	W	A	35	6.5	4.6
01-24-1994	05:50:24	34.36 N	118.63	W	A	39	12.1	4.3
01-24-1994	05:54:21	34.36 N	118.63	W	A	39	10.9	4.2
01-27-1994	17:19:58	34.27 N	118.56	W	A	28	14.9	4.6
01-28-1994	20:09:53	34.38 N	118.49	W	A	36	.7	4.2
01-29-1994	11:20:35	34.31 N	118.58	W	A	32	1.1	5.1
01-29-1994	12:16:56	34.28 N	118.61	W	A	31	2.7	4.3
02-03-1994	16:23:35	34.30 N	118.44	W	A	27	9.0	4.0
02-05-1994	08:51:29	34.37 N	118.65	W	A	41	15.4	4.0
02-06-1994	13:19:27	34.29 N	118.48	W	A	27	9.3	4.1
02-25-1994	12:59:12	34.36 N	118.48	W	A	34	1.2	4.0
03-20-1994	21:20:12	34.23 N	118.47	W	A	20	13.1	5.2
05-25-1994	12:56:57	34.31 N	118.39	W	A	28	7.0	4.4
06-15-1994	05:59:48	34.31 N	118.40	W	A	28	7.4	4.1
12-06-1994	03:48:34	34.29 N	118.39	W	A	26	9.0	4.5
02-19-1995	21:24:18	34.05 N	118.92	W	A	46	15.6	4.3
06-26-1995	08:40:28	34.39 N	118.67	W	A	44	13.3	5.0
03-20-1996	07:37:59	34.36 N	118.61	W	A	39	13.0	4.1
05-01-1996	19:49:56	34.35 N	118.70	W	A	42	14.4	4.1
10-23-1996	22:09:29	34.48 N	119.35	W	A	99	14.5	4.2
04-26-1997	10:37:30	34.37 N	118.67	W	A	42	16.5	5.1
04-26-1997	10:40:29	34.37 N	118.67	W	A	42	14.6	4.0

DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
04-27-1997	11:09:28	34.38 N	118.65 W	A	42	15.2	4.8
01-05-1998	18:14:06	33.95 N	117.71 W	A	66	11.5	4.3
08-20-1998	23:49:58	34.37 N	117.65 W	A	79	9.0	4.4
07-22-1999	09:57:24	34.40 N	118.61 W	A	42	11.6	4.0
03-07-2000	00:20:28	33.81 N	117.72 W	A	70	11.3	4.0

S E A R C H   O F   E A R T H Q U A K E   D A T A   F I L E 1

SITE: 2020 Ave of the Stars

COORDINATES OF SITE	34.0569 N 118.4138 W
DISTANCE PER DEGREE	110.9 KM-N 92.3 KM-W
MAGNITUDE LIMITS	4.0 - 8.5
TEMPORAL LIMITS	1932 - 2000
SEARCH RADIUS (KM)	100
NUMBER OF YEARS OF DATA	68.99
NUMBER OF EARTHQUAKES IN FILE	4136
NUMBER OF EARTHQUAKES IN AREA	400

L A W / C R A N D A L L

S E A R C H O F E A R T H Q U A K E D A T A F I L E 2

SITE: 2020 Ave of the Stars

COORDINATES OF SITE	34.0569 N 118.4138 W
DISTANCE PER DEGREE	110.9 KM-N 92.3 KM-W
MAGNITUDE LIMITS	6.0 - 8.5
TEMPORAL LIMITS	1906 - 1931
SEARCH RADIUS (KM)	100
NUMBER OF YEARS OF DATA	26.00
NUMBER OF EARTHQUAKES IN FILE	35
NUMBER OF EARTHQUAKES IN AREA	0

L A W / C R A N D A L L

LIST OF HISTORIC EARTHQUAKES OF MAGNITUDE 4.0 OR GREATER WITHIN 100 KM OF THE SITE
(NOAA/CDMG DATA 1812-1905)

DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
02-09-1890	04:06:00	34.00 N	117.50 W	D	85	.0	7.0

S E A R C H O F E A R T H Q U A K E D A T A F I L E 3

SITE: 2020 Ave of the Stars

COORDINATES OF SITE ...... 34.0569 N 118.4138 W

DISTANCE PER DEGREE ..... 110.9 KM-N 92.3 KM-W

MAGNITUDE LIMITS .... 7.0 - 8.5

TEMPORAL LIMITS .... 1812 - 1905

SEARCH RADIUS (KM) 100

NUMBER OF YEARS OF DATA 94.00

NUMBER OF EARTHQUAKES IN FILE 9

NUMBER OF EARTHQUAKES IN AREA 1

L A W / C R A N D A L L

S U M M A R Y   O F   E A R T H Q U A K E   S E A R C H

* * *

NUMBER OF HISTORIC EARTHQUAKES WITHIN 100 KM RADIUS OF SITE

MAGNITUDE RANGE	NUMBER
4.0 - 4.5	267
4.5 - 5.0	90
5.0 - 5.5	31
5.5 - 6.0	8
6.0 - 6.5	1
6.5 - 7.0	3
7.0 - 7.5	1
7.5 - 8.0	0
8.0 - 8.5	0

* * *

L A W / C R A N D A L L

C O M P U T A T I O N   O F   R E C U R R E N C E   C U R V E

L O G   N = A - B M

* * *

BIN	MAGNITUDE	RANGE	NO/YR (N)
1	4.00	4.00 - 8.50	5.79
2	4.50	4.50 - 8.50	1.92
3	5.00	5.00 - 8.50	.613
4	5.50	5.50 - 8.50	.163
5	6.00	6.00 - 8.50	.474E-01 NU
6	6.50	6.50 - 8.50	.369E-01
7	7.00	7.00 - 8.50	.529E-02 NU
8	7.50	7.50 - 8.50	.000
9	8.00	8.00 - 8.50	.000

A = .594   B = .4380   (NORMALIZED)
A = 4.283   B = .8942   SIGMA = .110

* * *

L A W / C R A N D A L L

C O M P U T A T I O N   O F   D E S I G N   M A G N I T U D E

C O N S T A N T   A R E A

* * *

TABLE OF DESIGN MAGNITUDES

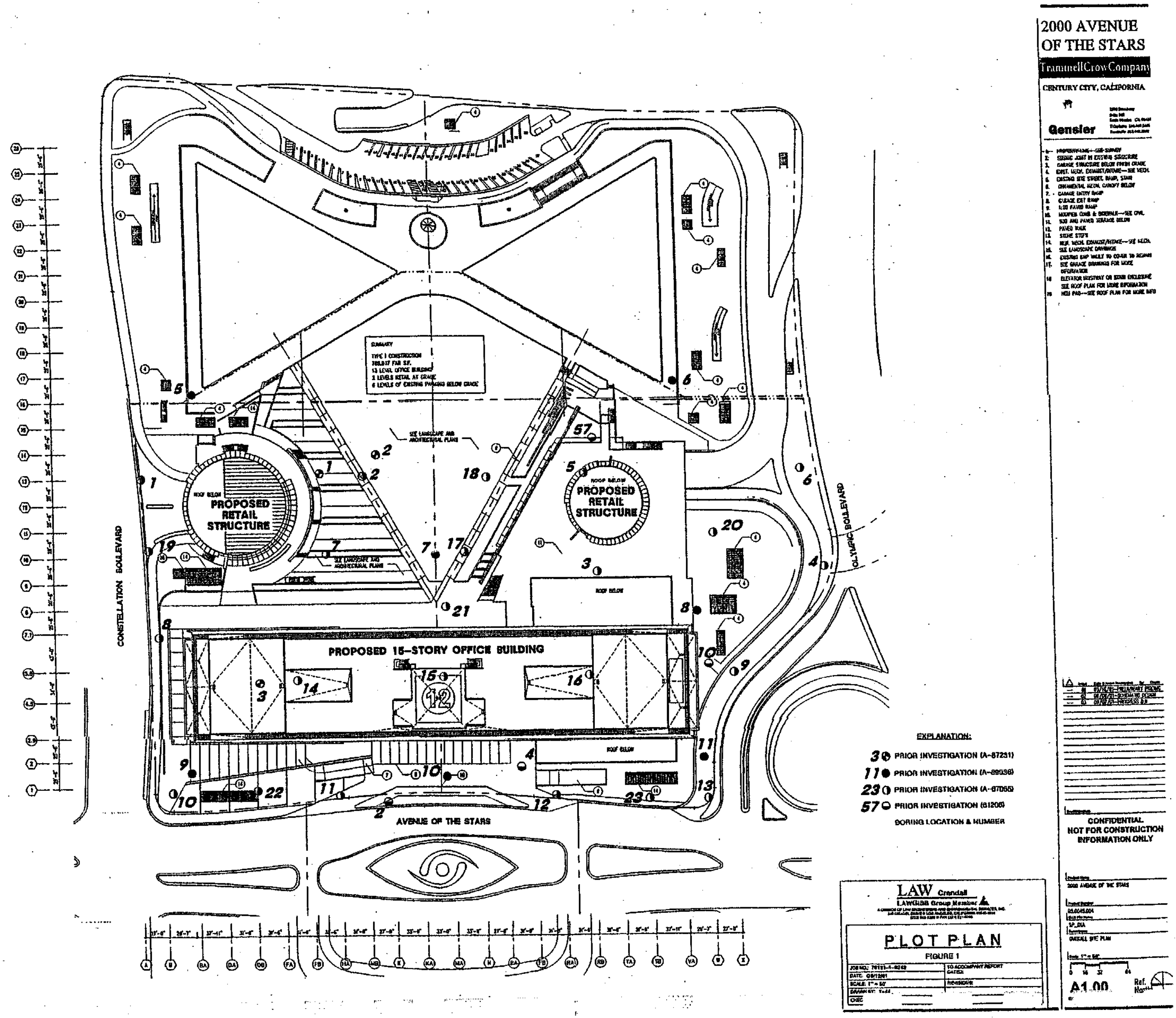
RISK		RETURN PERIOD (YEARS)					DESIGN MAGNITUDE			
		DESIGN LIFE (YEARS)								
		25	50	75	100		25	50	75	100
.01	..	2487	4974	7462	9949	..	8.22	8.34	8.39	8.42
.05	..	487	974	1462	1949	..	7.70	7.96	8.08	8.16
.10	..	237	474	711	949	..	7.40	7.69	7.85	7.95
.20	..	112	224	336	448	..	7.07	7.38	7.55	7.67
.30	..	70	140	210	280	..	6.85	7.17	7.35	7.48
.50	..	36	72	108	144	..	6.53	6.86	7.05	7.18
.70	..	20	41	62	83	..	6.27	6.60	6.79	6.93
.90	..	10	21	32	43	..	5.96	6.29	6.48	6.62

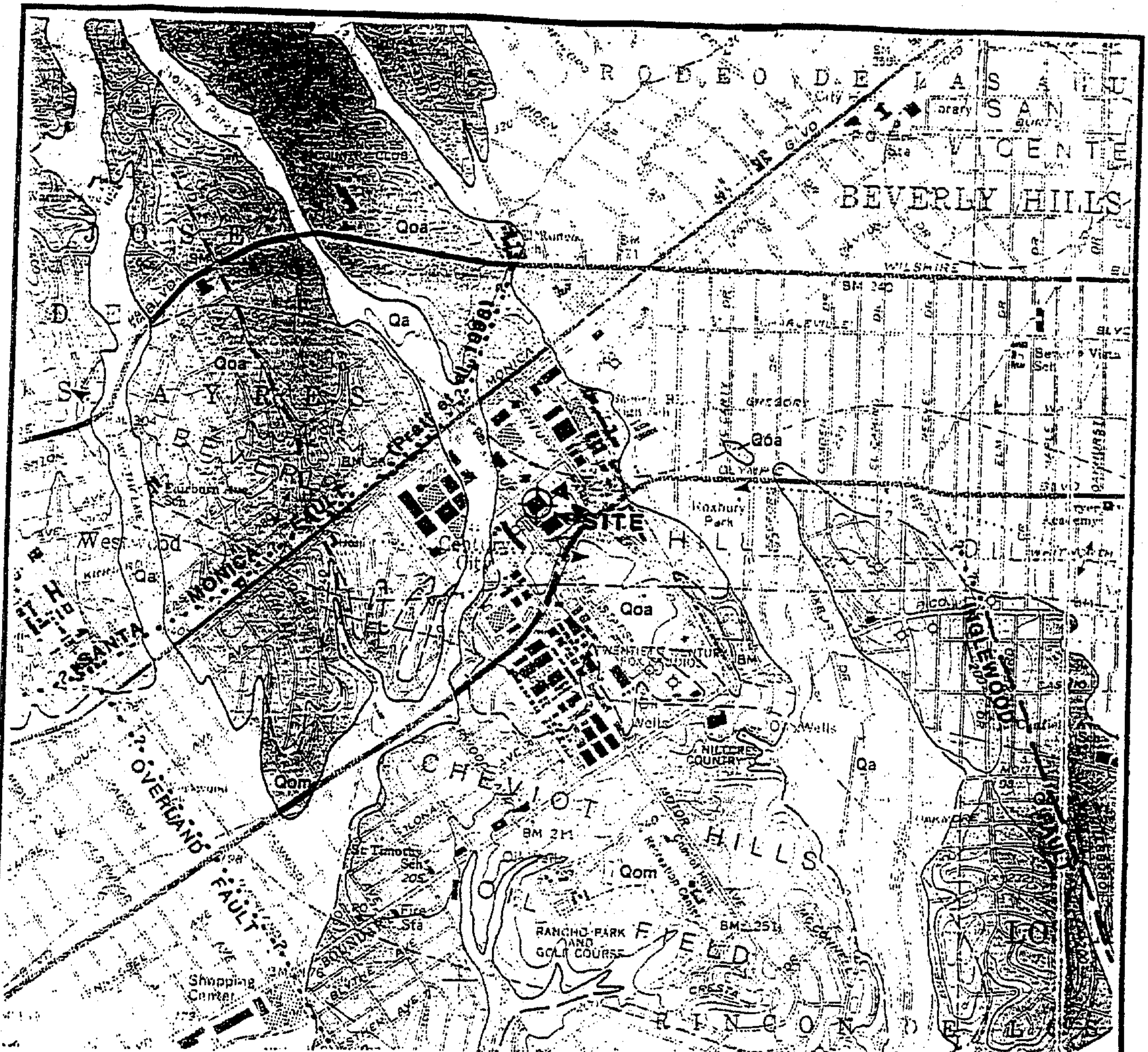
MMIN = 4.00 MMAX = 8.50
MU = 5.08 BETA = 2.059

* * *

L A W / C R A N D A L L

FIGURES


2000 AVENUE OF THE STARS
Trammell Crow Company
CENTURY CITY, CALIFORNIA
Gensler
PROPOSED 15-STORY OFFICE BUILDING
PROPOSED RETAIL STRUCTURE
ROOF BELOW
CONSTELLATION BOULEVARD
OLYMPIC BOULEVARD
AVENUE OF THE STARS
SEE LANDSCAPE AND ARCHITECTURAL PLANS
EXPLANATION:
3 PRIOR INVESTIGATION (A-87231)
11 PRIOR INVESTIGATION (A-89036)
23 PRIOR INVESTIGATION (A-87065)
57 PRIOR INVESTIGATION (61206)
BORING LOCATION & NUMBER
CONFIDENTIAL
NOT FOR CONSTRUCTION
INFORMATION ONLY
LAW Crandall
LAWGIBB Group Member
PLOT PLAN
FIGURE 1
A1.00



REFERENCE : Dibblee, T.W. Jr., 1991,
'Geologic Map of the Beverly Hills and Van Nuys
(South 1/2) Quadrangle, Los Angeles County, California.'

EXPLANATION :

Qa HOLOCENE age alluvium
Qoa PLEISTOCENE age older alluvium
Qom PLEISTOCENE age marine deposits

— — — GEOLOGIC CONTACT dashed where inferred or indefinite

·········· ANTICLINE dotted where concealed by surficial sediments

·?· — — FAULT dashed where approximate, dotted where concealed, queried where questionable

COORINATES :
34.0569N
118.4138W

N

LOCAL GEOLOGY

SCALE 1" = 2000' = 610 meters

LAW / CRANDALL

FIGURE 2

JOB 70131-1-0242 0001 DATE DR. GHC O.E. CHKD. SFK

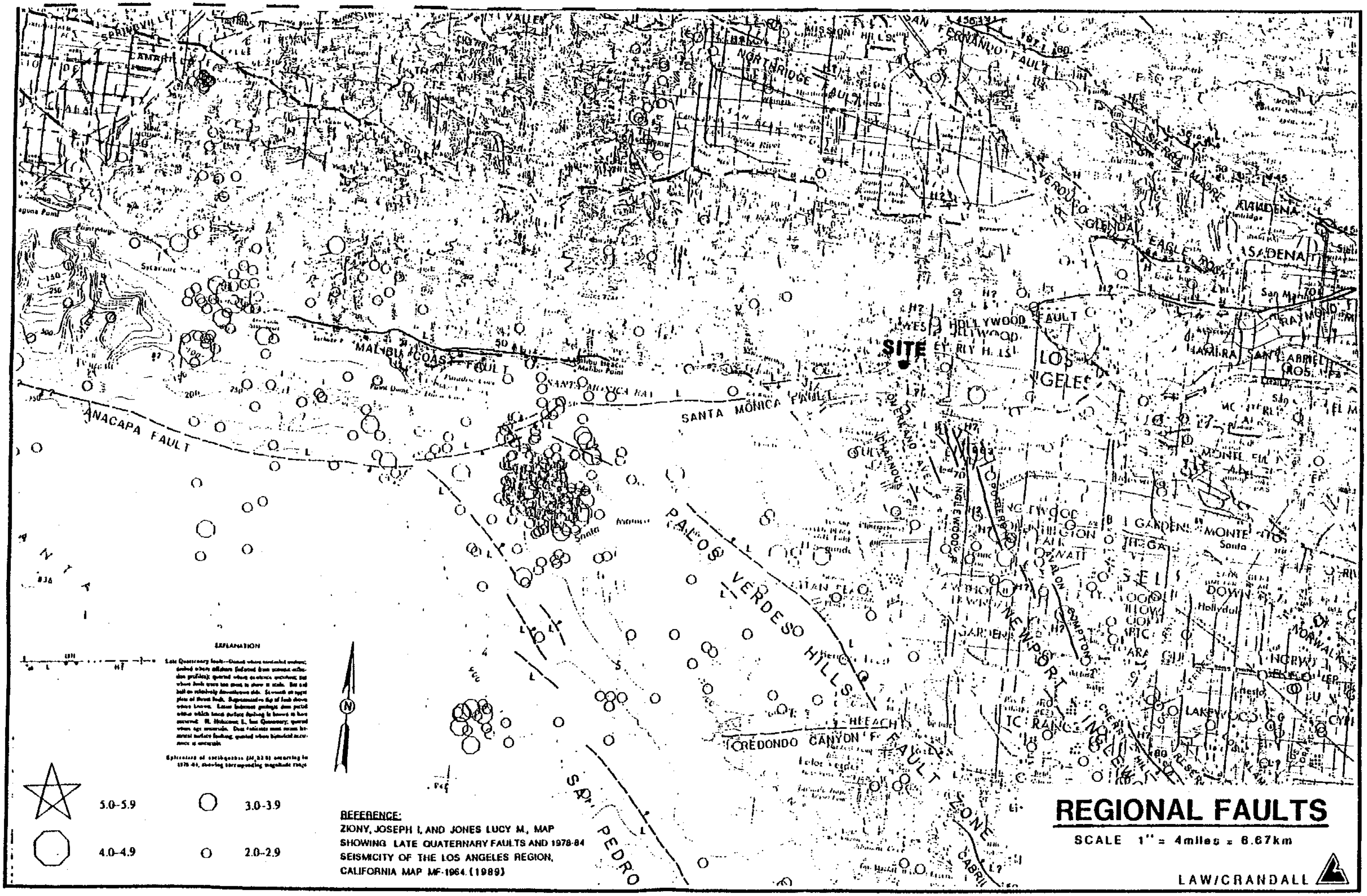
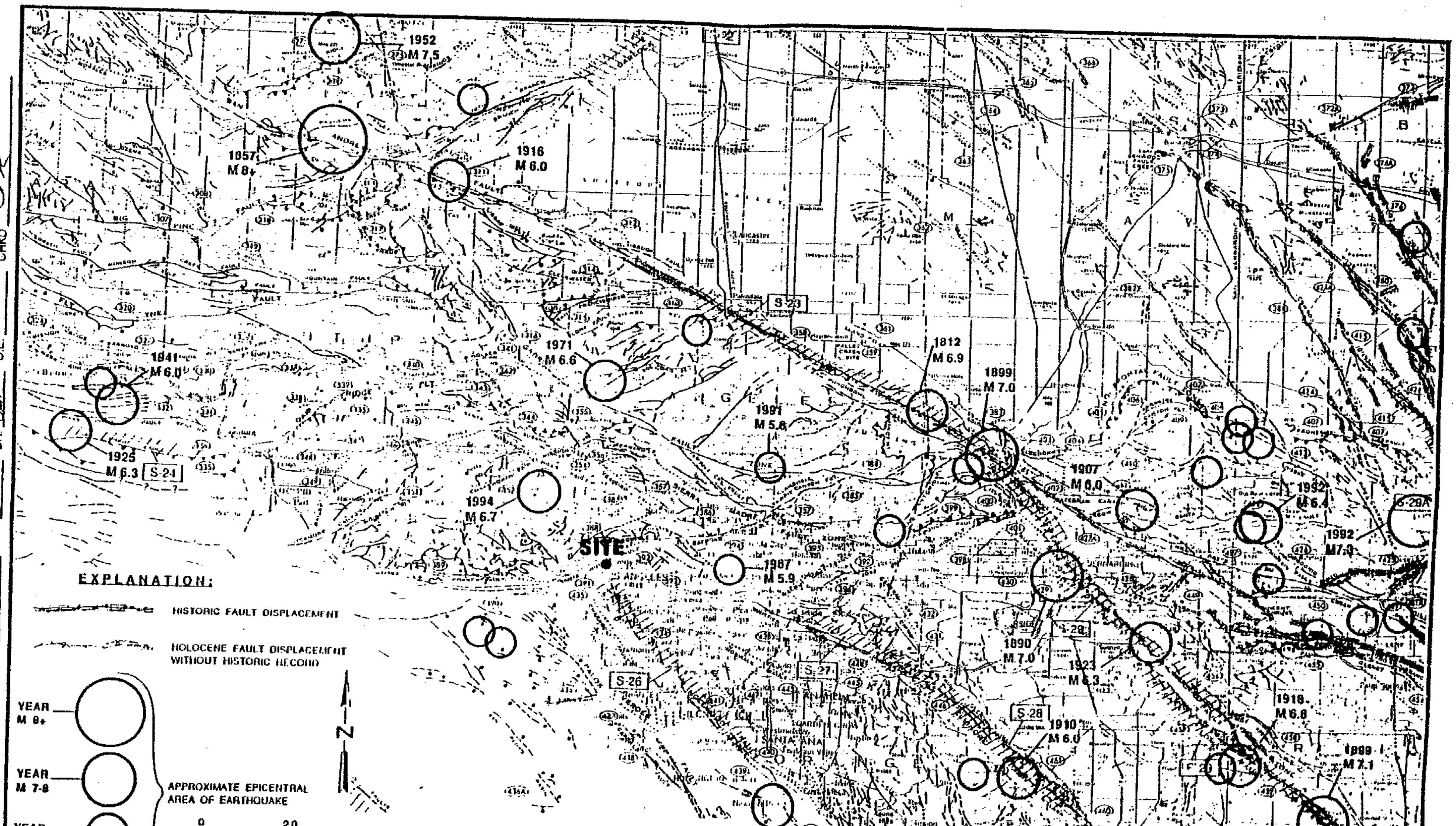

REGIONAL FAULTS
SCALE 1" = 4miles = 6.67km
LAW/CRANDALL
REFERENCE:
ZIONY, JOSEPH I, AND JONES LUCY M., MAP SHOWING LATE QUATERNARY FAULTS AND 1978-84 SEISMICITY OF THE LOS ANGELES REGION, CALIFORNIA MAP MF-1964. (1989)
EXPLANATION
5.0-5.9
4.0-4.9
3.0-3.9
2.0-2.9
SITE
SANTA MONICA FAULT
MALIBU COAST FAULT
ANACAPA FAULT
PALOS VERDES HILLS
SAN PEDRO
REDONDO CANYON
FAULT ZONE
NORTHRIDGE FAULT
MISSION HILLS
SAN FERNANDO FAULT
VERDUGO
EAGLE ROCK
RAYMOND
OVERLAND AVE
CHARNOCK
INGLEWOOD
POTRERO
COMPTON
CABRILLO

FIGURE 3



REFERENCES

1) JENNINGS, C.W., 1994, "FAULT ACTIVITY MAP OF CALIFORNIA AND ADJACENT AREAS WITH LOCATION AND AGES OF RECENT VOLCANIC ERUPTIONS", CALIFORNIA DIVISION OF MINES AND GEOLOGY, GDM 6.

2) EARTHQUAKE CATALOGS RICHTER, 1812-1932, NATIONAL OCEANIC ATMOSPHERIC ADMINISTRATION, 1812-1931, CALTECH, 1932-1997.

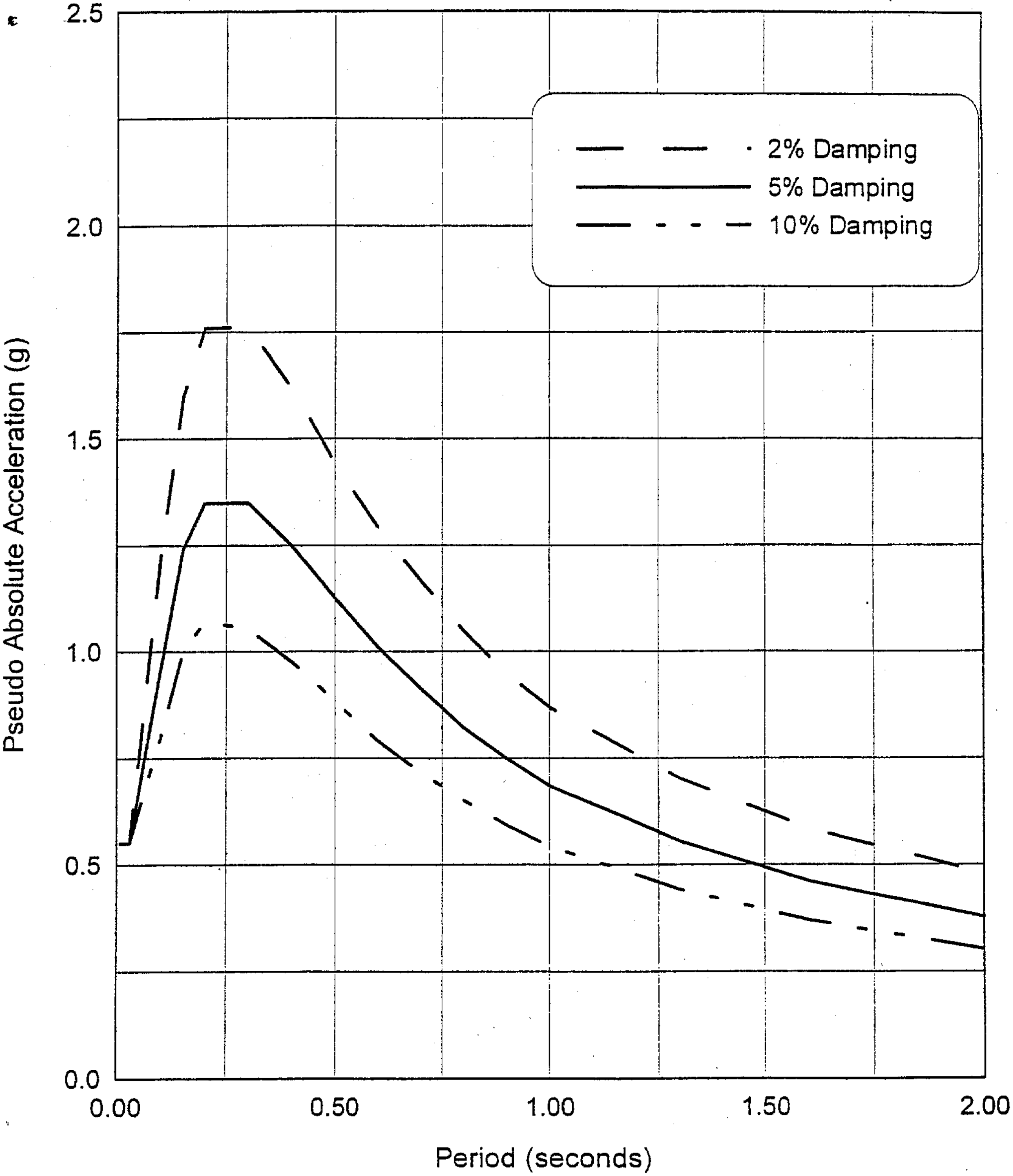
REGIONAL SEISMICITY

LAW/CRANDALL

JOB 70131-1-0242 DATE 7-20-01 F.T. DR. GMC O.E. CHKD

FORM 137-A

JOB: 70131-1-0242 DATE: 5 September 2001 F.T.: n/a DR.: TEA O.E.: cc CHKD: JM/CK



DESIGN SPECTRA

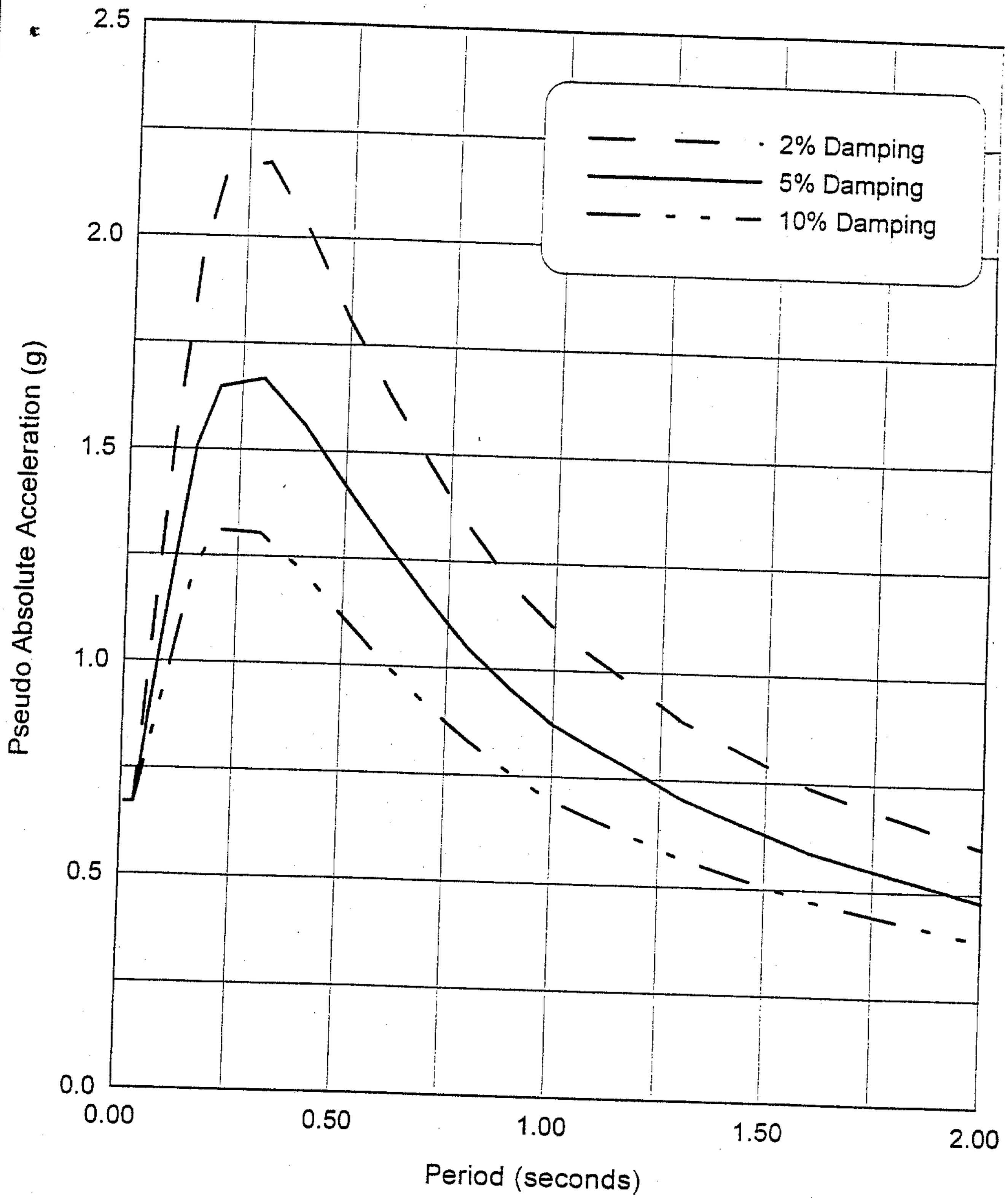
Design Basis Earthquake (DBE)

10% Probability of Exceedence in 50 Years

LAW/CRANDALL

FIGURE 5

g:\projects\70131\10242\Calculations and Ansiyses\Ground Motion\rspdbe.grf



DESIGN SPECTRA

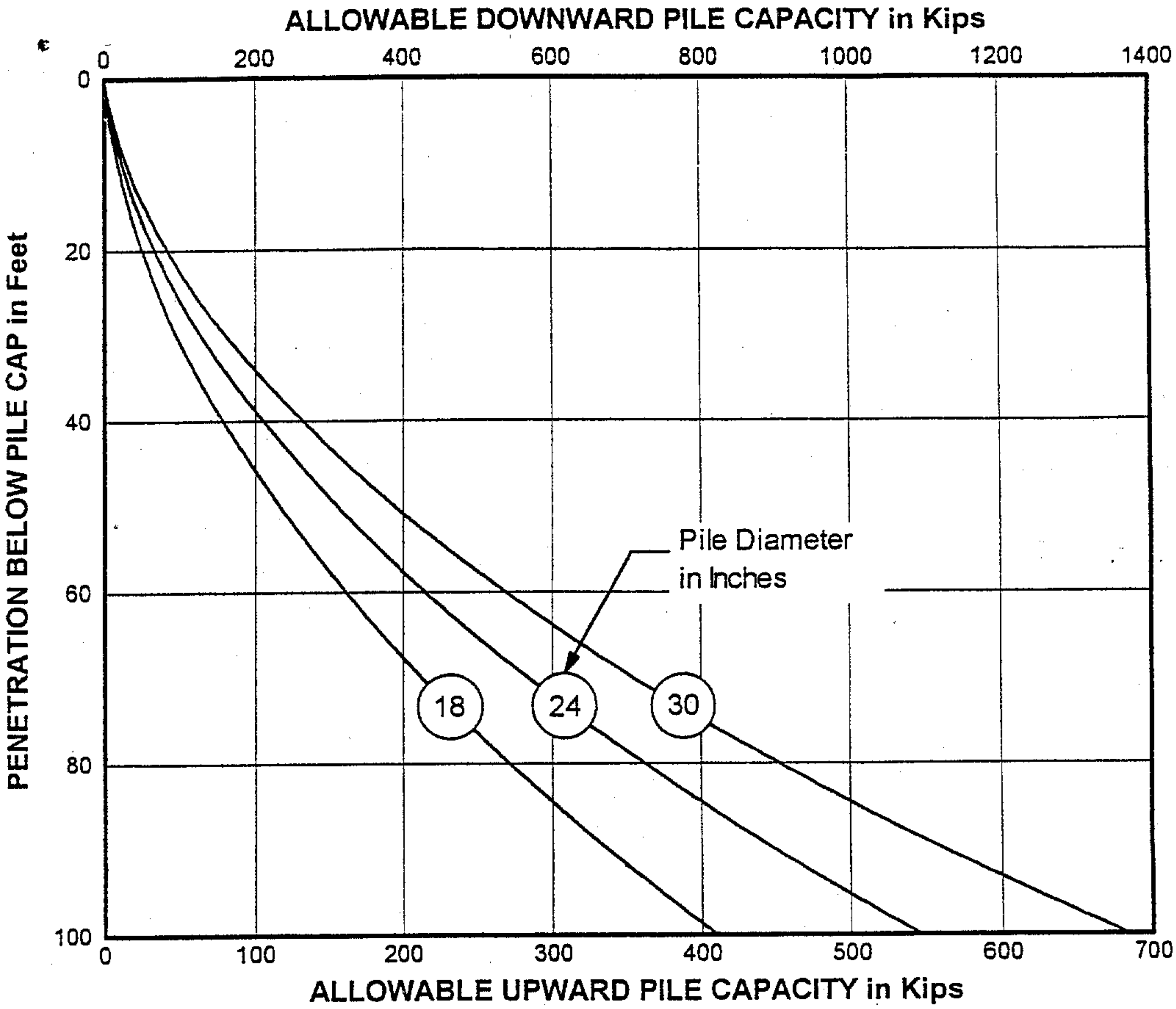
Maximum Capable Earthquake (MCE)
10% Probability of Exceedence in 100 Years

CHKD: JM/CK
O.E.: cc
DR.: TEA
F.T.: n/a
DATE: 5 September 2001
JOB: 70131-1-0242

LAW/CRANDALL

FIGURE 6

g:\projects\70131\10242\Calculations and Anslyses\Ground Motion\rspdbe.grf



JOB 70131-1-0242 DATE 10/5/01 DR MM OE ck CHKD

NOTES:

(1) The indicated values refer to the total of dead plus live loads; a one-third increase may be used when considering wind or seismic loads.

(2) Piles in groups should be spaced a minimum of 2-1/2 diameters on centers, and should be drilled and filled alternately with the concrete permitted to set at least 8 hours before drilling an adjacent hole.

(3) The indicated values are based on the strength of the soils; the actual pile capacities may be limited to lesser values by the strength of the piles.

DRILLED PILE CAPACITIES

LAW/CRANDALL

FIGURE 7