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June 1991

REPORT
GEOTECHNICAL AND GEOLOGIC/
SEISMIC HAZARD INVESTIGATION
PROPOSED FIRE STATION NO. 1
RELOCATION SITE
DONOHUE DRIVE
DUBLIN, CALIFORNIA
FOR
DOUGHERTY REGIONAL FIRE AUTHORITY

BSK
& Associates

BSK & Associates, Geotechnical Consultants, Inc

Geotechnical Engineering • Engineering Geology • Environmental Engineering • Engineering Laboratories • Chemical Laboratories

June 28, 1991 BSK JOB No. P91083

City of Dublin
P.O. Box 2340
Dublin, California 94568

Attention: Mr. Lee Thompson
Public Works Director

SUBJECT: Report - Geotechnical and Geologic/Seismic Hazard Investigation
Fire Station No. 1 Relocation Site
South Lot Adjoining 7494 Donohue Drive
Dublin, California
For Dougherty Regional Fire Authority

Gentlemen:

As requested and authorized, we have performed a geotechnical and geologic/seismic hazard investigation for the proposed Fire Station No. 1 Relocation Site in accordance with our Proposals PR91087, dated April 29, 1991 and PR91087A, dated May 17, 1991. The accompanying report contains our conclusions and recommendations with regard to site preparation and grading, foundation design, retaining structure design, interior concrete slab construction, and pavement design. This report also presents information on site geology, seismicity, and seismic hazards.

A preliminary geotechnical report, dated June 17, 1991, regarding the subject site, was submitted to the City of Dublin to comply with the project schedule contained in our agreement.

A separate report containing the results of our Preliminary Environmental Site Assessment for the subject project is concurrently being prepared for submittal under BSK Job No. P91082.

BSK & ASSOCIATES
1181 QUARRY LANE
BLDG. 300
PLEASANTON, CA 94566

[ ] Fresno, California 93706 • 1645 "E" Street, Suite 105 • Telephone (209) 485-3200, Fax (209) 485-7427
[ ] Fresno, California 93706 • 1445 "F" Street • Telephone (209) 485-0100, Fax (209) 268-7041
[ ] Fresno, California 93706 • 1414 Stanislaus Street • Telephone (209) 485-8310, Fax (209) 485-6935
[ ] Visalia, California 93291 • 808 E. Douglas Avenue • Telephone (209) 732-8857, Fax (209) 732-6570
[ ] Bakersfield, California 93304 • 117 "V" Street • Telephone (805) 327-0671, Fax (805) 324-4218
[X] Pleasanton, California 94566 • ~~5720 E Sonoma Drive~~ • Telephone (415) 462-4000, Fax (415) 462-6283
[ ] Sacramento, California 95827 • 9901 Horn Road, Suite C • Telephone (916) 363-1871, Fax (916) 363-1875

We appreciate the opportunity to be of service to you on this project. Should you have questions or comments regarding the contents of this report, please contact us.

Respectfully submitted,

BSK & ASSOCIATES

Alex Y. Eskandari

Alex Y. Eskandari, P.E.
Project Manager
C.E. 38101

REGISTERED PROFESSIONAL ENGINEER
YAHYA ESKANDARI
No. C038101
Exp. 3/31/93
CIVIL
STATE OF CALIFORNIA

Edward J. Uhlir

Edward J. Uhlir
C.E. 35598/C.E.G. 1275
916 - 363-1871

AYE/EJU/TWB:kl

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SOUTH LOT ADJOINING 7994 DONOHUE DRIVE
DUBLIN, CALIFORNIA

INTRODUCTION

General

This report presents the results of our geotechnical investigation performed for the proposed Fire Station No. 1 Relocation Site in Dublin, California and contains information regarding the geology and seismic hazards at the site. The subject site is located south of the existing fire station at 7994 Donohue Drive and northwest of the intersection of Amador Valley Boulevard and Donohue Drive. The site location, with respect to surrounding properties and streets, and the site layout with the locations of our exploratory borings, are shown on Figure 1, Vicinity Map, and Figure 2, Site Plan, respectively.

The report is based on review of available geotechnical and geologic data pertinent to the subject site and vicinity, and a field exploration program of four exploratory borings and a variety of laboratory tests. This report includes the field and laboratory investigation data and presents geotechnical conclusions and recommendations with regard to earthwork, foundation design, retaining structure design, interior concrete slab construction, and pavement design. This report also presents information on site geology, seismicity, and seismic hazards.

Planned Construction

Based on the information provided to BSK by the City of Dublin, we understand that the approximately 13,600 square foot lot will be developed as a new fire station to replace the existing fire station located north of the subject site. The new fire station will consist of a two-story building with a footprint of 2400 square feet and a 3150 square foot single-story building as shown on Figure 1, Site Plan. *The existing fire station building* will be demolished and replaced with a paved parking lot. The City of Dublin has informed us that an underground gasoline tank was abandoned in 1965 or 1966 by grouting in-place at the existing fire station and that two underground tanks were removed in November 1989. We do not have information or knowledge of the type of backfill material used or the adequacy of backfill placement.

At the time this report was prepared, information regarding the type of construction, building loads, and site grading was not available. We have assumed for the purpose of preparing this report that the buildings will utilize either wood frame, masonry block, or concrete tilt-up construction and the pad elevation will not vary by more than one to two feet from the ground elevation existing at the time of our field work (June 1991). We have also assumed that column loads will be in the range of 40 to 50 kips. We do not know if underground structures (basements, vaults, etc.) or retaining structures will be constructed at the site. This information should be reviewed by BSK & Associates when it becomes available to determine if the assumptions made during the preparation of this report remain valid.

In the event that changes occur in the nature or design of the project, the conclusions and recommendations contained in our report will not be considered valid unless the changes are reviewed and the conclusions of our report are modified or verified in writing.

Purpose and Scope

As outlined in our Proposals PR91087, dated April 29, 1991, and PR91087A, dated May 17, 1991, the purpose of this report is to describe the subsurface conditions encountered during field exploration and to provide geotechnical engineering recommendations for site preparation and earthwork procedures, foundation design parameters, retaining structure design parameters, interior concrete slab construction, and pavement design. This report also presents information on site geology, seismicity, and seismic hazards. The scope of our investigation included a program of field exploration, laboratory testing, geologic and engineering analysis, and preparation of this report.

Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices. This warranty is in lieu of all other warranties, either expressed or implied.

FIELD EXPLORATION AND LABORATORY TESTING

Field Exploration

Our field investigation consisted of a site surface reconnaissance and subsurface exploration. Four exploratory borings were drilled on June 3 and 4, 1991 at the locations shown on Figure 2, Site Plan. The borings were advanced with an 8-inch diameter hollow-stem auger from a truck-mounted Mobile B-53 drilling rig. The test borings were extended to depths of 21.5 to 51.5 feet below the existing grade.

The materials encountered in the borings were visually classified in the field, and logs were recorded by our engineer at that time. Visual classification of the materials encountered in our borings was generally made in accordance with the Unified Soil Classification System (ASTM D2487). A key for classification of the soils is presented on the Legend for Test Hole Logs, Figure 3. The logs of the borings are presented on Figures 4 through 7.

Samples of the subsurface materials were obtained by driving a 2.4-inch inside diameter, 3-1/4 inch outside diameter Modified California sampler and a 1-3/8 inch inside diameter split spoon sampler. Penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30 inch free fall to drive the sampler to a maximum depth of 18 inches. The number of blows required to drive the last 12 inches is recorded as Penetration Resistance (blows/foot) on the Boring Logs.

Soil samples were obtained from the exploratory borings at selected depths appropriate to the soil investigation. The samples were returned to our laboratory for further evaluation and testing.

Laboratory Testing

Laboratory testing of samples was conducted to determine the in-situ moisture and density, compressibility characteristics, shear strength, R-value, expansion index, and classification of the selected soil samples. The laboratory test data are presented on the attached Tables 1 through 6 and on the related figures.

SITE CONDITIONS

Surface/Site Description

The site investigated is currently occupied by an asphalt parking lot in the eastern half of the site and by an open field in the western half. The existing paved area consists of approximately 1 to 2 inches of asphalt concrete underlain by 8 to 10 inches of aggregate base at the locations of our borings. The site was relatively flat at the time of our field work (June 1991) except for several small piles of soil and rock in the open field. The open field had a sparse to moderate growth of dry grass and weeds at the time of our field work and irrigated landscaping consisting of bushes and shrubs existed around the parking lot. An existing storm drain utility line, approximately 3 to 4 feet below the top of asphalt, exists in the paved parking lot beneath proposed building locations. The site is bound by Donohue Drive to the east, the existing fire station to the north, Greenwood Apartments to the west, and by existing office and retail buildings to the south.

Soil Conditions

The results of our four exploratory borings, drilled to a maximum depth of 51.5 feet, indicate that the upper 29 feet of soil comprises stiff clay with varying percentages of silt and sand. Our laboratory tests indicate that the clay in the upper 5 feet at the site is potentially expansive. The clay is underlain by thinly interbedded layers of sand, silty sand, and clayey sand between approximate depths of 29 and 37 feet as encountered at the location of Boring 1. The interbedded layers are underlain by silty sand approximately 6 feet thick from 37 to 43 feet and by sandy clay and sandy silt from 43 to 51.5 feet, at Boring 1.

The soil profile described above is generalized; therefore, the reader is advised to consult the Logs of Borings (Figures 4 through 7 in the Appendix) if the soil conditions at a specific location are desired. On these logs, we have indicated the soil type, color, moisture content, dry density, and the applicable Unified Soil Classification System symbol.

The locations of our exploratory borings shown on Figure 1 were determined by taping from features shown on the site plan we were provided. Hence, accuracy can be implied only to the degree that this method warrants. Surface elevations at the boring locations were not determined since we were not provided with a topographic survey.

Groundwater

Groundwater was initially encountered at depths of 14 to 15 feet during drilling of the borings. Groundwater was measured at depths between 8 and 12 feet beneath the surface in our bore holes at the end of drilling (June 4, 1991). The borings were backfilled with cement grout after drilling. A history of groundwater fluctuations at the site is beyond the scope of this report. However, fluctuation of the groundwater level may occur due to seasonal rainfall, temperature, groundwater withdrawal, construction activities on this or adjacent properties, and other factors not evident at the time of our investigation.

GEOLOGY AND SEISMIC HAZARDS

General

This portion of our report addresses the geology and seismic hazards of the proposed fire station relocation site. Information presented in this section is based on a review of currently available literature and the results of our exploratory borings. Our evaluation of the seismic hazards at the site is based on information about known Bay Area faults and current state-of-the-art methods of analysis. Our scope of work did not include excavation of exploratory trenches at the site or the performance of geophysical surveys.

Site Geology

The site is underlain by Quaternary alluvial deposits consisting of unconsolidated to weakly consolidated clay, silt, sand and gravel. The deposits are less than 2 million years old and are typically more than 150 feet thick. A deep exploratory boring performed by others (Peter Kaldveer and Associates, 1987) indicates that the thickness of the alluvial deposits in the Dublin area may be as great as 275 feet. For the purposes of this study, we have assumed the thickness of the alluvial deposits to be between 150 and 300 feet. Determination of the alluvial deposit thickness at the site by exploratory boring is beyond the scope of our work for this report.

Fault Location and Seismicity

There are no known faults crossing or within one-quarter mile of the site. The site is not located within an Alquist-Priolo Special Studies Zone. However, several active faults exist within 30 miles of the site and strong ground shaking should be expected to occur at the site in association with large magnitude earthquakes along the nearby active faults. Active faults that could produce strong ground shaking at the site include the San Andreas located 28 miles to the west, the Hayward located 10 miles to the west, the Greenville located 7 miles to the east, and the Calaveras located within 1/4-mile west of the site (see Figure 8, Fault Location Map). Several other faults exist within 50 miles of the site, but most are not considered to be active or are considered to be active but are generally thought to be less capable of producing large magnitude earthquakes than the four aforementioned faults. The San Andreas, Hayward, Greenville, and Calaveras Faults are considered to be the most likely faults to generate strong seismic activity during the lifetime of the proposed structure. They have had seismic activity associated with them within historic time.

Maximum earthquake magnitudes are generally presented as either Maximum Credible Earthquake magnitudes or as Maximum Probable Earthquake magnitudes. The Maximum Credible Earthquake (MCE) is the maximum earthquake that appears capable of occurring under the presently known tectonic framework. It is a rational and believable event that is in accord with known geologic and seismologic facts. Little regard is given to its probability of occurrence, except that its likelihood of occurring is great enough to be of concern. The Maximum Probable Earthquake (MPE) is the maximum earthquake that is likely to occur during a 100-year interval. The postulated magnitude of the MPE should not be lower than the maximum that has occurred within historic time. The MCE and MPE of the faults considered in this study have been evaluated by others (Greensfelder 1974, Mualchin and Jones 1986, Slemmans and Chang 1982, Wright et al., 1982) and are presented in the following table.

FAULT	MCE	MPE
San Andreas	8.3	8.3
Hayward	7.5	7.0
Greenville	6.8	6.0
Calaveras	7.0	6.5

Ground Shaking

The intensity of ground shaking at a given site due to a seismic event is related to the magnitude of the earthquake, the nature of the material underlying the site (soil or rock), and the distance the site is from the focus of the earthquake. Ground accelerations will be attenuated by both bedrock and by soil overlying bedrock.

Peak horizontal accelerations in rock for different magnitude earthquakes as a function of the distance from the site to the energy release may be estimated by attenuation relationships modeled from empirical data. An attenuation relationship devised by Campbell in 1981 appears to best describe ground surface response to an earthquake near the project site, based on assumptions and limitations of similar relationships by others, and borne out by correlative data obtained from the Loma Prieta earthquake of 1989.

Based on the relationships described, the MCE and MPE of the San Andreas, Hayward, Greenville and Calaveras faults, and the distance of these faults to the project site, we have estimated the maximum surface horizontal acceleration at the project site due to seismic events along these faults to be 0.67g.

Ground Rupture and Fault Creep

Our review of available literature did not disclose the existence of known fault crossing the project site. We did not observe evidence of faulting at the project site during our field work. Therefore, it is our opinion that the risk of damage to structures due to ground rupture or fault creep along a fault at the site is small.

Liquefaction

Liquefaction may be defined as the process by which specific soil deposits lose their shear strength during earthquake-induced shaking which may result in excessive settlement of structures at the ground surface. Liquefaction is primarily associated with the buildup of pore pressure in loose, saturated, cohesionless sandy soil.

The results of our exploratory borings indicate that the soil conditions in the upper 20 to 29 feet at the boring locations consist of stiff to very stiff cohesive soil. We did not observe loose, cohesionless, sandy soil in the upper 20 to 29 feet at our boring locations. Thinly interbedded layers of sandy soil were encountered at the location of Boring 1 between depths of approximately 29 and 37 feet. Blow counts required to drive the sampler into the interbedded sandy soil layers indicate that the sandy soil is dense. Laboratory grain size analyses performed on samples of the sandy soil indicate the soil samples had 45% fines (silt and clay) content. Sandy soil was also encountered at the location of Boring 1 between approximate depths of 37 and 43 feet. Blow counts indicated that the soil at a depth of 37 feet is moderately dense, and a laboratory grain size analysis of a soil sample obtained from 37 feet indicated 21% fines content. The sandy soil is underlain by very stiff sandy clay and moderately dense sandy silt.

It is our opinion that the sandy soil between approximate depths of 29 and 37 feet at the location of Boring 1 has a low potential for liquefaction due to the percentage of fines in the soil and the soil density based on blow counts. However, a state-of-the-art estimation of the cyclic stress ratio required to initiate liquefaction of the sandy soil between depths of 37 and 43 feet indicates there is a potential for liquefaction between those depths. Our evaluation was based on an estimation of horizontal ground accelerations as described previously and upon the results of our exploratory borings and laboratory tests.

Although our evaluation of the soil between 37 and 43 feet indicates a potential for liquefaction, it is our opinion that the risk of damage to proposed structures due to liquefaction of soil between 37 and 43 feet is small due to the nature of the overlying soil. Studies performed by BSK for ValleyCare Hospital and by Peter Kaldveer and Associates for the Dublin Civic Center (1987) concluded that liquefaction damage to structures was unlikely.

Seismic Induced Landsliding

Slope failure due to earthquake activity is not a consideration due to the flat surface of the site and surrounding area.

Seismic Settlement

Differential settlement of loose soil beneath a structure due to ground shaking is a cause of damage during earthquakes. Vibration settlement of improperly compacted soil can occur even during moderate earthquakes. Seismic settlement would pose little risk at this site if the native soil and fill are compacted in accordance with the recommendations of this report.

Lateral Spreading

Lateral spreading is typically associated with liquefied soil flowing on very low slopes. It is our opinion that the risk of lateral spreading occurring at the site is small based on the results of our exploratory borings and the flat surface of the site and surrounding area.

Other Seismic Hazards

Other seismic hazards, such as seismically-induced flooding, tsunamis, and seiches are not likely at the site since the factors that cause such hazards do not exist at the site.

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CONCLUSIONS AND RECOMMENDATIONS

General

Based upon the data collected during this investigation, and from a geotechnical engineering standpoint, it is our opinion that the site is suitable for the proposed fire station relocation. The proposed buildings may be supported on shallow foundations, provided that the recommendations presented herein, and conclusions and recommendations contained in the Preliminary Environmental Site Assessment Report (P91082) for the subject site, dated June 1991, are incorporated in the design and construction of the project.

Based on the results of our laboratory tests, the clay in the upper 5 feet at the site exhibits moderate expansion potential as determined by an Expansion Index Test. The primary geotechnical consideration at the site is the potentially expansive soil. We recommend that the moisture content of the near-surface soil be maintained at a moisture content 3 to 4 percent above optimum until placement of slabs-on-grade and foundations.

Detailed geotechnical engineering recommendations are presented in the remaining portions of the text, and are based on the properties of the materials encountered during our investigation.

Site Preparation and Earthwork

Stripping and Clearing. Prior to commencement of the site grading, the site should be stripped and cleared of surface vegetation and organic-laden topsoil (2-3 inches). Materials resulting from clearing and stripping operations should be removed from the site; however, organic topsoil and surface strippings can be stockpiled, if desired, for reuse later as topsoil in future landscaped areas. These materials should not be used for engineered fill. Asphalt and concrete associated with the existing paved area in the eastern half of the site should be completely removed from the site. Aggregate baserock beneath the existing asphalt may be utilized as fill in proposed pavement areas if it is thoroughly mixed with soil. The aggregate base should be completely removed from all building areas to a distance of 5 feet beyond the perimeter building lines. The existing storm drain utility line beneath proposed structures should be removed to a distance of at least ten feet beyond the perimeter building lines. The resulting depression should be backfilled in accordance with the following paragraphs. We are not aware of other existing utility lines in the proposed building locations. However, if other utility lines exist, they should also be removed.

Preparation of Building Areas. After the existing storm drain utility line (and other potential utility lines) is removed as recommended in the previous paragraph, the excavation should be widened if necessary to allow access with mechanical compaction equipment and all loose soil should be removed. Subsequent backfill should be placed and compacted in accordance with the following paragraphs. After stripping and clearing operations are completed, the exposed subgrade in the building area should be scarified to a depth of at least 8 inches, brought to a moisture content approximately 3 to 4 percent higher than optimum, and mechanically compacted to the requirements for engineered fill, described below. The limits of scarification and recompaction should extend at least to five feet beyond the boundaries of the buildings and should include exterior columns and sidewalk areas adjacent to the buildings.

Preparation of Pavement Areas. After the site clearing operations are complete, the proposed pavement areas must be properly prepared. Areas to receive fill should be scarified to a depth of eight inches, moisture conditioned to or near the optimum moisture content, and recompacted in accordance with the compaction recommendations of this report. Proposed pavement areas that are at subgrade elevation or that will be excavated to subgrade elevation should be scarified and recompacted similarly to areas receiving fill.

Material for Fill. The site soils are suitable for use as borrow material with the exception of the organic-laden topsoil. Over-wet or saturated soils, if any, should not be used as engineered fill unless allowed to dry to a moisture level 3 to 4 percent higher than optimum. If additional fill is necessary to attain the finish grade, it may be selectively obtained from the over-excavation of the pavement areas. Soil used as fill should not contain rock larger than 4 inches in any dimension, debris, or other deleterious material. Aggregate base beneath the existing paved area in the eastern half of the site may be used as fill in proposed pavement areas (but not in building areas) provided it is free of deleterious material and is mixed with native soil. Import material (if needed) should be reviewed by BSK prior to transportation to the site.

Fill Placement. Native soil used as fill should be moisture-conditioned to 3 to 4 percent above the optimum moisture content prior to compaction. Fill material that has excessive moisture should be allowed to dry prior to compaction or be mixed with dry soil to bring the fill to a workable moisture content. Fill should be placed in level lifts not exceeding a loose, uncompacted thickness of 8 inches.

Compaction. The scarified subgrade and subsequent structural fill placed at the site should be compacted to at least 90 percent. The upper eight inches of subgrade in the pavement areas should be compacted to at least 95 percent. The terms "compacted and compaction" refer to relative compaction as determined by ASTM Test Designation D1557. We recommend that compaction be performed by mechanical means only.

Utility Trench Backfill

Utility trench backfill should be placed in accordance with the compaction requirements and procedures for engineered fill and the additional recommendations presented in this section. Small diameter underground utility lines should have no less than 12 inches of cover. A minimum of six inches of compacted sand bedding, extending six inches above the pipe, should be provided. The remaining utility trench may be backfilled with the on-site soils, with the exception of topsoil. The sand bedding and cover should be compacted to 85 percent and the remaining on-site soil backfill should be placed in 8 inch lifts (loose) and mechanically compacted, 2 to 3 percent above optimum moisture, to at least 90 percent. The pipe bedding factors used for design should take into account the recommended degree of compaction.

Granular import material may be used as trench backfill provided that it is compacted to 95 percent relative compaction, and the upper one foot of the backfill consists of the on-site clayey soils. No jetting of backfill should be permitted.

Where utility trenches cross continuous perimeter footings, the granular fill should be terminated at least one foot on either side of the footing. This zone should be backfilled with on-site clayey material for the entire depth of the trench above the sand bedding.

Site Drainage

Site drainage should be carefully designed to efficiently carry surface water away from foundations and pavements to suitable discharge points. Continuous building roof gutters are recommended. Downspouts should be connected to closed pipes which discharge into the site storm drain system. Water should not be allowed to pond around the foundations.

Foundation Design Criteria

The proposed buildings may be supported on a shallow foundation system bearing on undisturbed native soil or engineered fill. The footings may be designed to impose a maximum allowable pressure of 2,500 pounds per square foot due to dead load and 3,500 pounds per square foot due to dead plus live load. These values may be increased by one-third for transient loads such as seismic or wind. The footings should extend to a depth of at least 24 inches below the surface of the building pad. Continuous strip footings should be at least 12 inches wide and isolated column footings should be at least 2 feet square. A perimeter concrete cut-off foundation system should be constructed to reduce the risk of moisture migration beneath the perimeter footings and into the soil beneath the interior concrete slab. The cut-off foundation system may consist of 24 inch deep continuous strip footings. If concrete tilt-up perimeter walls are proposed, the base of the walls could be designed to rest at least 24 inches below the lowest adjacent exterior grade.

Continuous perimeter footings should be reinforced with top and bottom steel to provide structural continuity and to permit spanning of local irregularities. Reinforcement should be designed by a structural engineer and should take into consideration pressures developed by expansive soil. Visible desiccation cracks in the bottoms of footing excavations should be closed by soaking prior to placement of concrete. We recommend that BSK be retained to observe the footing excavations prior to placing reinforcing steel or concrete in order to check that footings are founded in the anticipated bearing soil and that proper moisture conditions have been achieved in the soil. We also recommend that BSK be retained to observe placement of reinforcement and concrete and to obtain concrete samples for testing.

Lateral loads on foundations may be resisted by friction under the foundations and passive earth pressure on the sides. The coefficient of friction between the foundation and underlying soils may be assumed to be 0.35. The passive earth pressure provided by the foundation backfill may be assumed to be equal to that exerted by a fluid with a unit weight of 250 pounds per cubic foot (pcf). The passive pressures and friction factor given are ultimate values. Safety factors consistent with the design objectives should be incorporated. A minimum factor of safety of 1.5 against lateral sliding is recommended if the sliding is resisted only by frictional resistance. When combined passive and frictional resistance is used, we recommend a minimum safety factor of 2.0. For lateral stability against seismic loading, we recommend a minimum safety factor of 1.1.

In order to maintain adequate support for the foundations, footings located adjacent to utility trenches, including existing utility trenches or other footings, should be deepened as necessary so that their bearing surfaces are below an imaginary plane having an inclination of 1.5 horizontal to 1.0 vertical, extending upward from the bottom edge of the adjacent trench or footing.

The results of our laboratory tests indicate that total settlement is expected to be on the order of 0.5 to 1.0 inch with differential settlement between adjacent footings approximately 50% of the total settlement. This assumes that the recommendations of this report are followed and that column loads are on the order of 40 to 50 kips.

Interior Concrete Slab-on-Grade

Interior concrete slab-on-grade floors subjected to heavy floor loads such as those from fire trucks should be underlain by at least 6 inches of Class 2 Aggregate Base (Caltrans Standard Specification, Section 26, January 1988) compacted to at least 95 percent of the maximum dry density as determined by ASTM D1557. This layer should be placed between the finished soil subgrade and the floor slab. Thickness and reinforcement of slabs should be determined by a structural engineer and should consider not only floor loading, but also pressures developed by potentially expansive soil beneath the slab.

Interior concrete slab-on-grade floors that will be covered by moisture sensitive floor coverings (such as sleeping areas and office areas) and will be subjected to light loads, should be underlain by a layer of compacted, washed, crushed rock or gravel at least 4 inches in thickness. The rock should be graded so that 100% passes the 3/4-inch sieve and 0% to 5% passes the No. 4 sieve. Rock should be compacted with a minimum of two passes with a vibratory type compactor. The rock layer should be overlain by an impermeable vapor barrier at least 20 mils thick. The vapor barrier should be overlain by two inches of clean sand to protect the vapor barrier during construction and to aid in curing of the concrete. The sand should be lightly moistened prior to placing the concrete. Slab thickness and reinforcement should be determined by a structural engineer and should take into consideration expansive soil pressures.

The soil subgrade within slab-on-grade areas should be checked by BSK prior to placement of the baserock, crushed rock, or gravel layer. If the soil is observed to have dried since grading operations, we recommend that a moisture content of 3% to 4% over the optimum moisture content be re-established in the upper 12 inches of the subgrade soil. Soaking of the subgrade soil should be checked by BSK prior to placement of concrete.

Floor Closure Strip - Tilt-up Construction

Construction for "Tilt-up" type buildings customarily causes the loading of wall footings to be virtually instantaneous upon erection. In contrast, the main area of the floor, because it has been used as a casting surface, has been subjected to significant surcharge over a number of weeks. At the time the walls are lifted and placed on the footings and the closure strip is cast, immediate settlement of the wall footings takes place at the same time rebound of the unloaded slab is occurring.

This condition causes differential settlement across the width of the closure strip. This differential settlement substantially approaches the design settlement for the foundation system.

To avoid undesirable differential settlement across the closure strip and opening of cold joints, the strip should be independent of the wall and foundation or should be cast at the latest possible time, in order to mitigate differential movement.

Retaining Structure Design Criteria

BSK was not aware of retaining structures at the site at the time we prepared this report. The following retaining structure design criteria is provided at the request of the City of Dublin. BSK should review plans of proposed retaining structures when the plans become available.

Soil pressures exerted against a retaining wall may be assumed to be equivalent to the pressure exerted by a fluid. Pressures developed by the equivalent fluid depend upon the design condition. The active design case should be used in computations for retaining structures which are free to rotate at the top and where slight wall movement away from the soil is possible. At-rest pressures should be utilized where a wall is restrained from moving at the top of the retaining structure and no wall movement is allowed. Passive earth pressures develop when structures move into the soil. The following equivalent fluid weights and coefficient of friction to resist sliding may be used for design of retaining structures if the backfill consists of properly compacted native soil and there are horizontal surfaces behind and in front of the retaining structure. The values given below do not include a factor of safety.

SUMMARY OF LATERAL EARTH PRESSURES	
Condition	Equivalent Fluid Pressure (pcf)
Active Pressure, Drained	45
At-Rest Pressure, Drained	65
Passive Pressure, Drained	250
Coefficient of Friction	0.35

The equivalent fluid pressures provided do not take into account sloping backfill or the effect of surcharge loading. Walls that have backfill that slope upward away from the wall should be designed for an additional fluid pressure of 1 pound per cubic foot for every 2 degrees of slope inclination. Backfill slopes should not be steeper than a 2:1 slope gradient (horizontal to vertical). The design wall pressure should also be increased if surcharge loads from footings, pavement, etc. exist within a distance from the wall equal to half the height of the wall, or if equipment loads will be applied near the wall. In general, walls subjected to surcharge loading should be designed for an additional uniform lateral pressure equal to one-half the anticipated surcharge load. BSK should review situations where sloping backfill or surcharge loading will occur at the top of a retaining wall prior to construction of the wall.

The equivalent fluid pressures provided assume drainage behind the retaining structure to prevent the build-up of hydrostatic pressures behind the wall. Drainage behind retaining structures may be provided by a vertical layer of pea gravel or crushed rock at least 12 inches in thickness positioned between the retaining wall and the soil backfill. The rock should be graded so that 100% passes the 3/4 inch sieve and 0% to 5% passes the No. 4 sieve. A geotextile filter fabric such as Mirafi 140 NS, Typar 3401, or the equivalent, should be placed between the rock drainage layer and the soil backfill to reduce the chances that the rock layer could become clogged with soil. A geosynthetic drainage composite such as Miradrain or Enkadrain may be substituted for the rock drainage layer. Care must be taken during installation to place the drainage composite so that the filter fabric of the composite faces the soil backfill and that the filter fabric is not torn or the composite crushed during construction. The drain should be placed directly behind the retaining wall, and extend from the base of the wall to within 1 foot of the top of the wall. Collected water may be removed either by installing weep holes along the bottom of the wall or by installing a perforated drainage pipe along the bottom of the permeable material. The drainage pipe should be Schedule 40 P.V.C. pipe at least 4 inches in diameter placed with perforations down and sloped at a maximum gradient of 1% away from the wall to suitable drainage intake facilities.

Pavement Design

A sample of soil in the upper 3 feet at the site was obtained during our field work and returned to our laboratory for R-value testing. The sample was tested in accordance with California Test 301. The results of the R-value test indicate an R-value of 21. We reduced the R-value to 10 for design purposes due to the expansion potential of the clay and possible variations in the soil subgrade.

We were not provided with traffic indices for the project. Therefore, we used a range of traffic indices for determining a range of pavement sections that may be applicable to the project. The project civil engineer or a traffic engineer should determine which pavement sections are applicable to the project based on anticipated traffic. The following table presents recommended pavement sections based on a design R-value of 10 for soil subgrade and a range of traffic indices.

TABLE I: ASPHALT PAVEMENT SECTIONS (THICKNESS IN INCHES)

Traffic Index	4.0	4.5	5.0	5.5	6.0
Asphalt Concrete	2.0	2.0	2.5	3.0	3.0
Aggregate Base	8.0	10.0	10.0	12.0	13.0

The pavement sections presented in Table I do not include the asphalt factor of safety as used by Caltrans. Therefore, the thicknesses shown in Table I should be used as minimum thicknesses, i.e., no reduction in thickness for construction tolerances should be allowed. The pavement sections are based on the assumption that the upper 8 inches of soil subgrade have been properly scarified, moisture conditioned to or at optimum moisture, and compacted to at least 95%, and that baserock is uniformly compacted to a minimum 95% relative compaction. The baserock (Class 2) is assumed to have a minimum R-value of 78. The Class 2 baserock should conform to the appropriate section of the Caltrans Standard Specifications. The soil subgrade must have a minimum R-value of 10. Import soil used as fill in pavement areas should be tested for R-value prior to transportation to the site. Relative compaction should be based on the ASTM D1557-78 test standard for the determination of maximum dry density. Pavement areas should be sloped at a gradient of 2% or greater to allow for positive surface drainage. Both the proper surface slope and uniform compaction are necessary for proper pavement performance.

Concrete pavement sections (if desired for the project) should consist of concrete at least 4 inches thick in areas subjected to automobile traffic only (i.e., no fire trucks) and at least 6 inches thick in areas subject to heavy truck traffic such as fire trucks. The Portland Cement Association recommends a minimum concrete compressive strength of 4000 psi for concrete pavement areas. The concrete pavement should be placed directly on uniformly compacted Class 2 baserock at least 6 inches thick. Baserock should be compacted to a minimum 95% relative compaction based on the ASTM D1557-78 test standard. The concrete pavement should be gradually thickened by 2 inches starting 2 feet from the edge for increased strength near the edges. Reinforcement of concrete pavement should be designed by the structural engineer. The soil subgrade should be prepared as recommended in the Site Preparation section of this report and in the previous paragraph.

With the relatively impermeable soil on the site, it is important that the drainage of pavement areas be carefully designed so that no water is allowed under them. If water is trapped under pavement, the water can fill the area between the soil and pavement, resulting in possible pavement failures. Where pavement abuts planter areas, the risk of water saturated subgrade failures can be reduced by using a concrete cut-off curb to minimize water from the planter areas from migrating under the pavement into the baserock. The cut-off curb should be 4 inches wide and extend at least 6 inches into the soil subgrade beneath the baserock.

Plans and Specifications

We recommend that a review of plans and specifications with regard to foundations and earthwork be performed by our office staff prior to the start of construction.

Construction Testing and Observation

We recommend that BSK be retained to provide testing and observation services during the site preparation and grading, and foundation construction phases of the project. This is to observe compliance with the design concepts, specifications and recommendations, and to provide consultation as required during construction.

CHANGED CONDITIONS AND LIMITATIONS

The analyses and recommendations submitted in this report are based upon the data obtained from the exploratory borings performed at the locations shown on the Site Plan, Figure 1.

The report does not reflect variations which may occur between borings. The nature and extent of such variations may not become evident until construction is initiated. If variations then appear, a re-evaluation of the recommendations of this report will be necessary after performing on-site observations during the time of site clearing and site preparation and noting the characteristics of any variations.

The validity of the recommendations contained in this report is also dependent upon an adequate testing and observation program during the construction phase. Our firm assumes no responsibility for construction compliance with the design concepts or recommendations unless we have been retained to perform the on-site testing and review during construction.

The findings of this report are valid as of the present. However, changes in the conditions of the site can occur with the passage of time, whether caused by natural processes or the work of man, on this property or adjacent property. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation, governmental policy or the broadening of knowledge. Therefore, this report should be reviewed to determine the applicability of the conclusions and recommendations considering changed conditions or after a substantial lapse of time between the preparation of our report and the start of work at the site (2 years or more).

BSK & Associates has prepared this report for the exclusive use of the Owner and the Project Design Consultants. The report has been prepared in accordance with generally accepted practices using the degree of care ordinarily exercised under similar circumstances, by reputable geotechnical engineers and geologists practicing in this or a similar locality. No other warranties, either expressed or implied, are made as to the professional advice provided under the terms of this agreement and included in this report.

BSK & Associates

SUMMARY OF LABORATORY TEST DATA

TABLE 1

SUMMARY OF ATTERBERG LIMITS TEST DATA

Test Boring Number	Sample Depth (Feet)	Liquid Limit (percent)	Plasticity Index (percent)	Sample Classification (U.S.C.S.)
B-1	3.0	48	26	CL/CH
B-3	2.5	30	12	CL

TABLE 2

SUMMARY OF GRAIN-SIZE ANALYSES DATA

Test Boring Number	Sample Depth (Feet)	Percent Passing #200 Sieve	Soil Classification (U.S.C.S.)
B-1	30-1/2	34	SM
B-1	35-1/2	45	SM
B-1	37-1/2	22	SM
B-1	50-1/2	62	ML

TABLE 3

SUMMARY OF UNCONFINED COMPRESSION TEST DATA

Test Boring Number	Sample Depth (Feet)	In-Situ Moisture (percent)	Dry Density (pcf)	Unconfined Compressive Strength (psf)	Axial Strain (percent)
B-1	8	28	94	4084	6.1
B-4	3	18	107	3925	6.1

SUMMARY OF LABORATORY TEST DATA (CONTINUED)

TABLE 4

SUMMARY OF CONSOLIDATION TEST DATA

Test Boring Number	Sample Depth (Feet)	In-Situ Moisture (percent)	Dry Density (pcf)	Consolidation* Characteristics	Consol. Figure Number
B-3	6	18.4	109	1.5% < 2.8 ksf	9

(*) Slope per cycle below pre-consolidation pressure.

TABLE 5

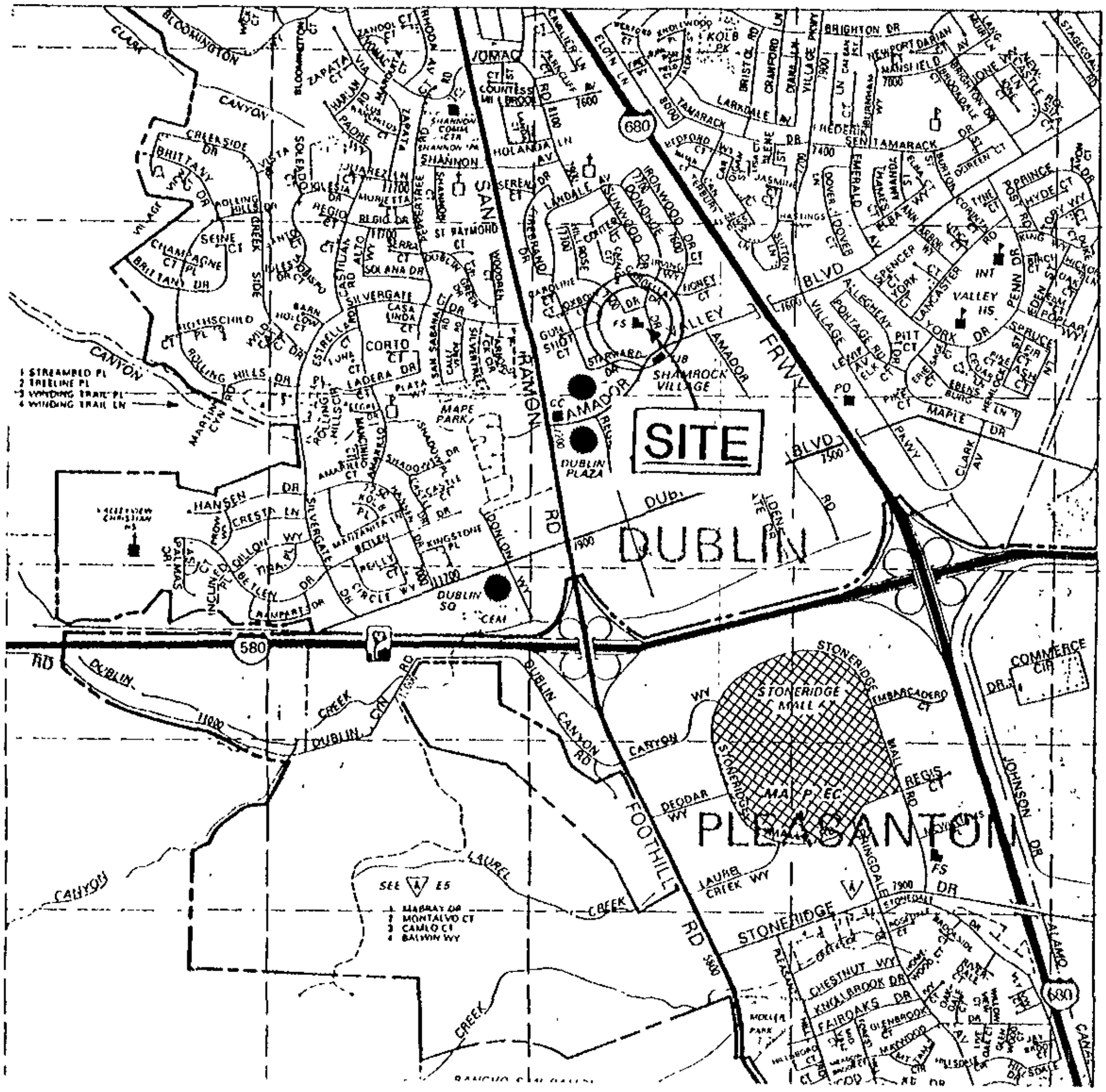
SUMMARY OF R-VALUE TEST DATA

Test Boring Number	Sample Depth (Feet)	R-value at 300 psi Exudation Pressure (%)	R-value Diagram Number
B-2, B-3 & B-4 (Composite)	0-3	21	11

TABLE 6

SUMMARY OF EXPANSION INDEX TEST DATA

Test Boring Number	Sample Depth (Feet)	In-Situ Moisture (percent)	Dry Density (pcf)	Expansion Index	Expansion Potential
B-2, B-3 and B-4 Composite	0-3	10.6	106.4	55	Medium*



VICINITY MAP

GEOTECHNICAL INVESTIGATION

FIRE STATION NO. 1 RELOCATION SITE

DONOHUE DRIVE

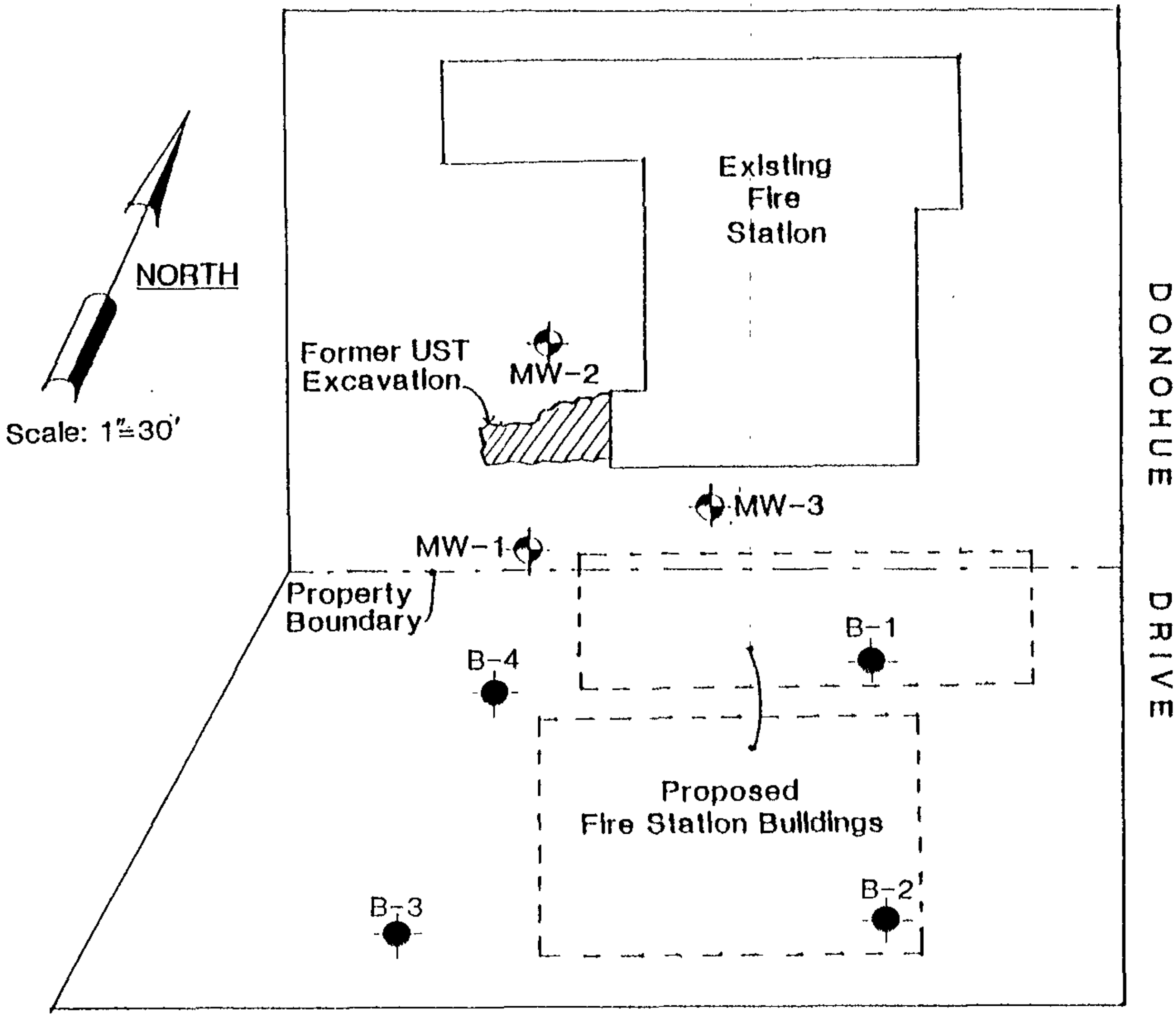
DUBLIN, CALIFORNIA

BSK Job No. P91083
June 1991
FIGURE: 1

BSK & Associates

BY TWB DATE 6/26/91 CHECKED BY AYE

BY EAU DATE 6-25-91 CHECKED BY AYE



LEGEND:

- Previously Installed Groundwater Monitoring Well
- Geotechnical Exploration Boring Drilled for this Investigation

SITE PLAN

GEOTECHNICAL INVESTIGATION
FIRE STATION NO. 1 RELOCATION SITE
DUBLIN, CALIFORNIA

BSK Job No. P91083
June 1991
FIGURE: 2

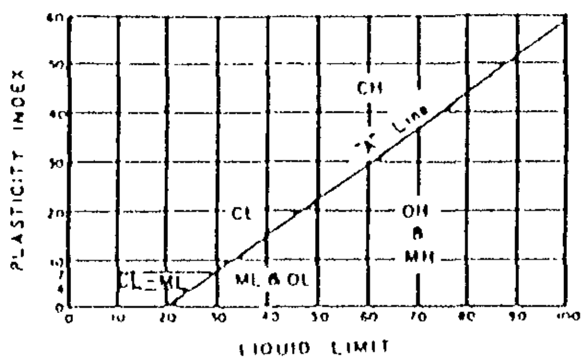
BSK & Associates

FIGURE: 3

LEGEND FOR TEST HOLE LOGS

METHOD OF SOIL CLASSIFICATION
(Unified Soil Classification System)

MAJOR DIVISIONS			SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 1/2 of soil > no 200 sieve size)	GRAVELS (More than 1/2 of coarse fraction > no 4 sieve size)	GW		Well graded gravels or gravel-sand mixtures, little or no fines
		GP		Poorly graded gravels or gravel-sand mixtures, little or no fines
		GM		Silty gravels, gravel-sand-silt mixtures
		GC		Clayey gravels, gravel-sand-clay mixtures
	SANDS (More than 1/2 of coarse fraction < no 4 sieve size)	SW		Well graded sands or gravelly sands, little or no fines
		SP		Poorly graded sands or gravelly sands, little or no fines
		SM		Silty sands, sand-silt mixtures
		SC		Clayey sands, sand-clay mixtures
FINE GRAINED SOILS (More than 1/2 of soil < no. 200 sieve size)	SILTS & CLAYS LL < 50	ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
		CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		OL		Organic silts and organic silty clays of low plasticity
	SILTS & CLAYS LL > 50	MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH		Inorganic clays of high plasticity, fat clays
		OH		Organic clays of medium to high plasticity, organic silty clays, organic silts



PLASTICITY CHART

Key to Samples

- Indicates depth of undisturbed sample
- Indicates depth of disturbed sample
- Indicates depth of Standard Penetration Split Spoon Sample
- Sample not recovered

BSK & Associates

DATE: 6/03/91

LOGGED BY: EJU

ELEVATION: ---

WATER LEVEL: 15' during drilling & rose to 12' at completion of drilling

EQUIPMENT: Mobile B-53, 8" Hollow Stem Auger

LOG DESIGNATION B-1

JOB: P91083

FIGURE: 4

DEPTH, FEET	NOMINAL (1) DIAMETER, IN.	BLOWS/FOOT (2)	MOISTURE %	DRY DENSITY, PCF	SAMPLES	U.S.C.S.	SOIL OR ROCK DESCRIPTION	NOTES
						PMT	1.5" Asphalt Concrete and 8" Aggregate Base	
						CL	SANDY CLAY: Dark brown, moist, stiff to very stiff, some fine gravel	
	2.5	27	21	104	1	CL/CH	SILTY CLAY: Dark gray, moist, stiff	PP* = 1.75 tsf OVM** to 1.2
5								
	2.5	33	--	--	2		Grades very stiff	PP - 1.75 tsf OVM to 12
	2.5	29	28	94	3			PP = 2.25 tsf OVM to 7
							Grades stiff to firm	PP = 1.25 tsf
10	2.5	28	30	91	4			
								▽ Water level after drilling
15								▽ (Initial
	2.5	32	20	109	5			PP - 2.25 tsf
						CL	SANDY CLAY: Brown, very moist, medium stiff to stiff	
20								PP = 1.0 tsf
	2.5	29	24	103	6			
						CL	SILTY CLAY: Medium brown-gray, very moist, stiff	
25								

THE LOGS SHOW SUBSURFACE CONDITIONS AT THE DATES AND LOCATIONS INDICATED, AND IT IS NOT WARRANTED THAT THEY ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES

(1) SAMPLER INSIDE DIAM.

(2) 140lb HAMMER - 30 INCH DROP.

(P) HYDRAULICALLY PUSHED

BSK & Associates

LOG DESIGNATION B-1 (Cont'd.)

DATE: 6/03/91
LOGGED BY: EJU
ELEVATION: ---
WATER LEVEL 15' during drilling & rose to 12' at completion of drilling
EQUIPMENT: Mobile B-53, 8" Hollow Stem Auger

JOB: P91083
FIGURE: 4 (contd)

DEPTH, FEET	NOMINAL (1) DIAMETER, IN.	BLOWS/FOOT (2)	MOISTURE %	DRY DENSITY, PCF	SAMPLES	U.S.C.S.	SOIL OR ROCK DESCRIPTION	NOTES
25	2.5	26	23	104	7	CL	SILTY CLAY: Mottled brown-gray, very moist, stiff	
30	2.5	72	--	--	8	SM/SP/SC	Interbedded SILTY SAND/SAND/CLAYEY SAND: Brown, very moist to wet, dense, some small gravel	34% passing #200 sieve
35	2.5	71	--	--	9			45% passing #200 sieve
	1 3/8	16	--	--	10	SM	SILTY SAND: Brown, wet, medium dense	SPT 22% passing #200 sieve
40								
							SANDY CLAY: Brown, very moist, very stiff	
45	1 3/8	24	--	--	11	CL		SPT
						ML	SANDY SILT: Brown, very moist, moderately dense	
50								

THE LOGS SHOW SUBSURFACE CONDITIONS AT THE DATES AND LOCATIONS INDICATED, AND IT IS NOT WARRANTED THAT THEY ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

(1) SAMPLER INSIDE DIAM.
(2) 140 lb HAMMER - 30 INCH DROP.
(P) HYDRAULICALLY PUSHED

BSK & Associates

DATE: 6/03/91

LOGGED BY: EJU

ELEVATION: ---

WATER LEVEL: 15' during drilling and rose to 12' at completion of drilling

EQUIPMENT: Mobile B-53, 8" Hollow Stem Auger

LOG DESIGNATION B-1 (Cont'd.)

JOB: P91083

FIGURE: 4 (cont'd)

DEPTH, FEET	NOMINAL (1) DIAMETER, IN.	BLOWS/FOOT (2)	MOISTURE %	DRY DENSITY, PCF	SAMPLES	U.S.C.S.	SOIL OR ROCK DESCRIPTION	NOTES
50	$1\frac{3}{8}$	13	--	--	12	ML	SANDY SILT: Brown, very moist, moderately dense	SPT
								Boring terminated at 51.5 feet. Backfilled with grout.
55								

Notes

* PP Denotes Unconfined Compressive Strength by Pocket Penetrometer

** OVM Denotes Organic Vapor Meter Reading

THE LOGS SHOW SUBSURFACE CONDITIONS AT THE DATES AND LOCATIONS INDICATED, AND IT IS NOT WARRANTED THAT THEY ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

(1) SAMPLER INSIDE DIAM.

(2) 140lb HAMMER - 30 INCH DROP.

(P) HYDRAULICALLY PUSHED

BSK & Associates

DATE: 6/04/91

LOGGED BY: EJU

ELEVATION: ---

LOG DESIGNATION B-2

WATER LEVEL: 14' during drilling and rose to 8' at completion of drilling

EQUIPMENT: Mobile B-53, 8" Hollow Stem Auger

JOB: P91083

FIGURE: 5

DEPTH, FEET	NOMINAL (1) DIAMETER, IN.	BLOWS/FOOT (2)	MOISTURE %	DRY DENSITY, PCF	SAMPLES	U.S.C.S.	SOIL OR ROCK DESCRIPTION	NOTES
							1" Asphalt Concrete and 10" Aggregate Base	
						CL	SANDY CLAY: Dark brown, moist, very stiff some fine gravel	Bulk Sample
	2.5	30	--	--	1	CL/CH	SILTY CLAY: Dark gray to brown, moist to very moist, stiff to very stiff	PP* = 1.75 tsf
5	2.5	29	19	111	2			PP = 2.0 tsf
							Grades to stiff	Water level after drilling
10	2.5	27	31	90	3		Trace of fine sand at 10 feet	PP = 1.25 tsf
						CL	SILTY CLAY: Mottled brown-tan, very moist, some fine sand, very stiff	(Initial)
15	2.5	46	--	--	4			PP = 2.25 tsf
20	2.5	26	24	102	5		Sand percentage increases with depth	
							Note * PP = Denotes Unconfined Compressive Strength by Pocket Penetrometer	Boring terminated at 21.5 feet. Backfilled with grout.
25								

THE LOGS SHOW SUBSURFACE CONDITIONS AT THE DATES AND LOCATIONS INDICATED, AND IT IS NOT WARRANTED THAT THEY ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

(1) SAMPLER INSIDE DIAM.

(2) 140lb HAMMER - 30 INCH DROP.

(P) HYDRAULICALLY PUSHED

BSK & Associates

DATE: 6/04/91

LOGGED BY: EJU

ELEVATION: ---

WATER LEVEL: 14.5' during drilling and rose to 9.5' at completion of drilling

EQUIPMENT: Mobile B-53, 8" Hollow Stem Auger

LOG DESIGNATION B-3

JOB: P91083

FIGURE: 6

DEPTH, FEET	NOMINAL (1) DIAMETER, IN.	BLOWS/FOOT (2)	MOISTURE %	DRY DENSITY, PCF	SAMPLES	U.S.C.S.	SOIL OR ROCK DESCRIPTION	NOTES
						CL	SANDY CLAY: Dark brown, moist, very stiff, some fine gravel	Bulk Sample
	2.5	21	18	107	1			
5						CL/CH	SILTY CLAY: Dark gray to brown, moist, very stiff, trace of fine sand	PP* = 2.0 tsf
	2.5	37	18	109	2			
	2.5	37	16	115	3			PP = 3.0 tsf
10								Water level after drilling
	2.5	27	28	94	4		Grades firm to stiff	PP = 0.75 tsf
						CL	SILTY CLAY: Mottled brown-gray, moist, stiff, some fine sand	(Initial)
15	2.5	40	18	113	5			PP = 1.5 tsf
							Sand percentage increases with depth, grades into Sandy Clay	
20								
	2.5	26	22	106	6			PP = 1.25 tsf
							Note * PP Denotes Unconfined Compressive Strength by Pocket Penetrometer	Boring terminated at 21.5 feet. Backfilled with grout.

THE LOGS SHOW SUBSURFACE CONDITIONS AT THE DATES AND LOCATIONS INDICATED, AND IT IS NOT WARRANTED THAT THEY ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS

(1) SAMPLER INSIDE DIAM.

(2) 140# HAMMER - 30 INCH DROP.

(P) HYDRAULICALLY PUSHED

BSK & Associates

DATE: 6/04/91
LOGGED BY: EJU
ELEVATION: ---
WATER LEVEL: 14' during drilling and rose to 9.5' at completion of drilling
EQUIPMENT: Mobile B-53, 8" Hollow Stem Auger

LOG DESIGNATION B-4

JOB: P91083
FIGURE: 7

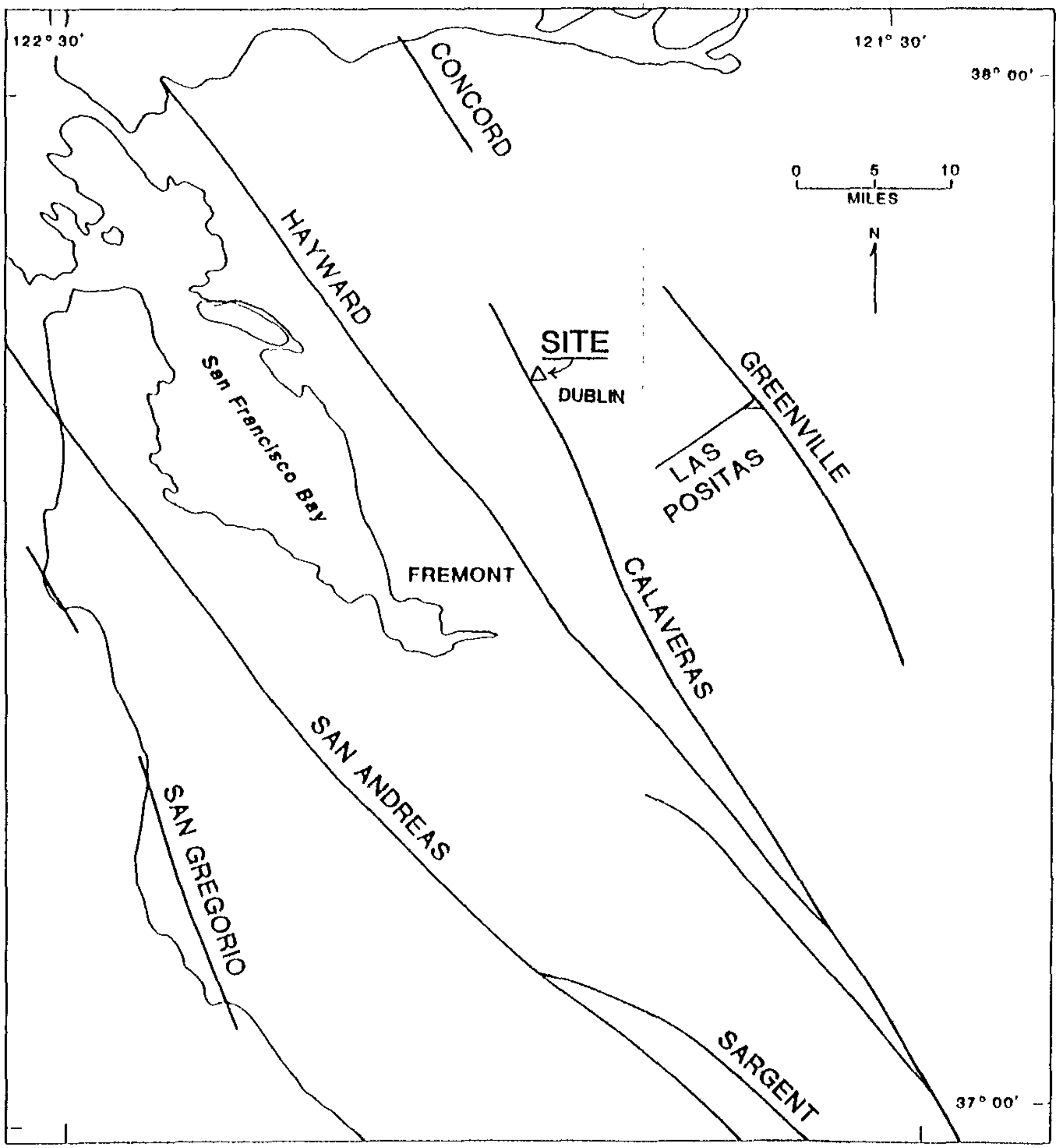
DEPTH, FEET	NOMINAL (1) DIAMETER, IN.	BLOWS/FOOT (2)	MOISTURE %	DRY DENSITY, PCF	SAMPLES	U.S.C.S.	SOIL OR ROCK DESCRIPTION	NOTES
								Bulk Sample
	2.5	25	18	107	1	CL/CH	SILTY CLAY: Dark gray to brown, moist to very moist, stiff, trace of fine sand	PP* = 1.75 tsf OVM = 0
5								
	2.5	39	18	111	2			OVM** = 0
10								Water level after drilling
	2.5	29	28	91	3			PP = 1.5 tsf OVM = 0
						CL	SILTY CLAY: Brown-gray, very moist, stiff to very stiff	
15								
	2.5	45	18	112	4			PP = 2.5 tsf
20								
	2.5	26	22	105	5			PP = 1.25 tsf
							Notes PP* Denotes Unconfined Compressive Strength by Pocket Penetrometer OVM** Denotes Organic Vapor Meter Ready	Boring terminated at 21.5 feet. Backfilled with grout.
25								

THE LOGS SHOW SUBSURFACE CONDITIONS AT THE DATES AND LOCATIONS INDICATED, AND IT IS NOT WARRANTED THAT THEY ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

(1) SAMPLER INSIDE DIAM.
(2) 140 lb HAMMER - 30 INCH DROP.
(P) HYDRAULICALLY PUSHED

BSK & Associates

CHECKED BY ______ DATE ______ BY ______



FAULT LOCATION MAP

BSK Job No. P91083
June 1991
FIGURE: 8

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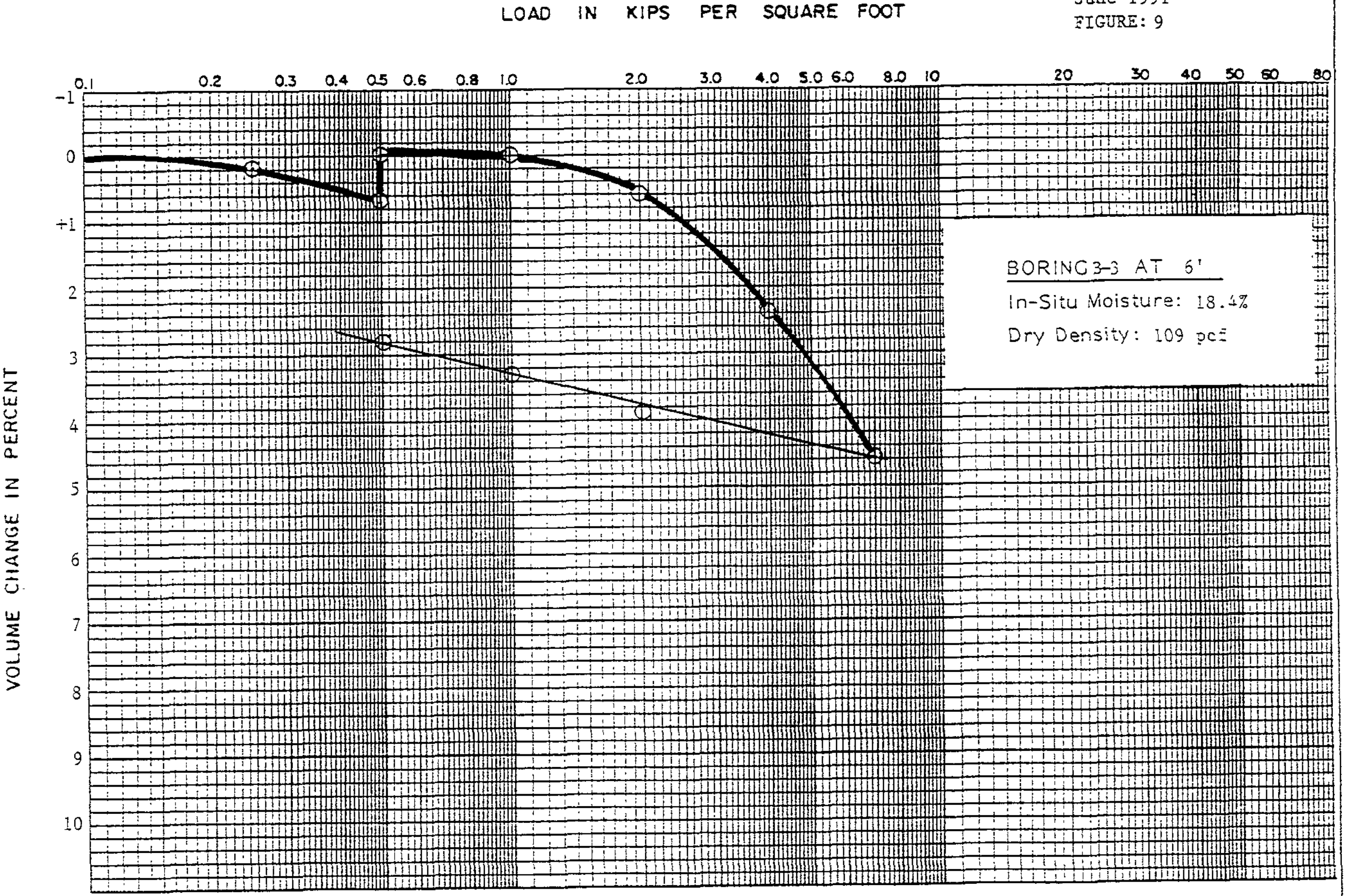
Job No. P91083
June 1991
FIGURE: 9

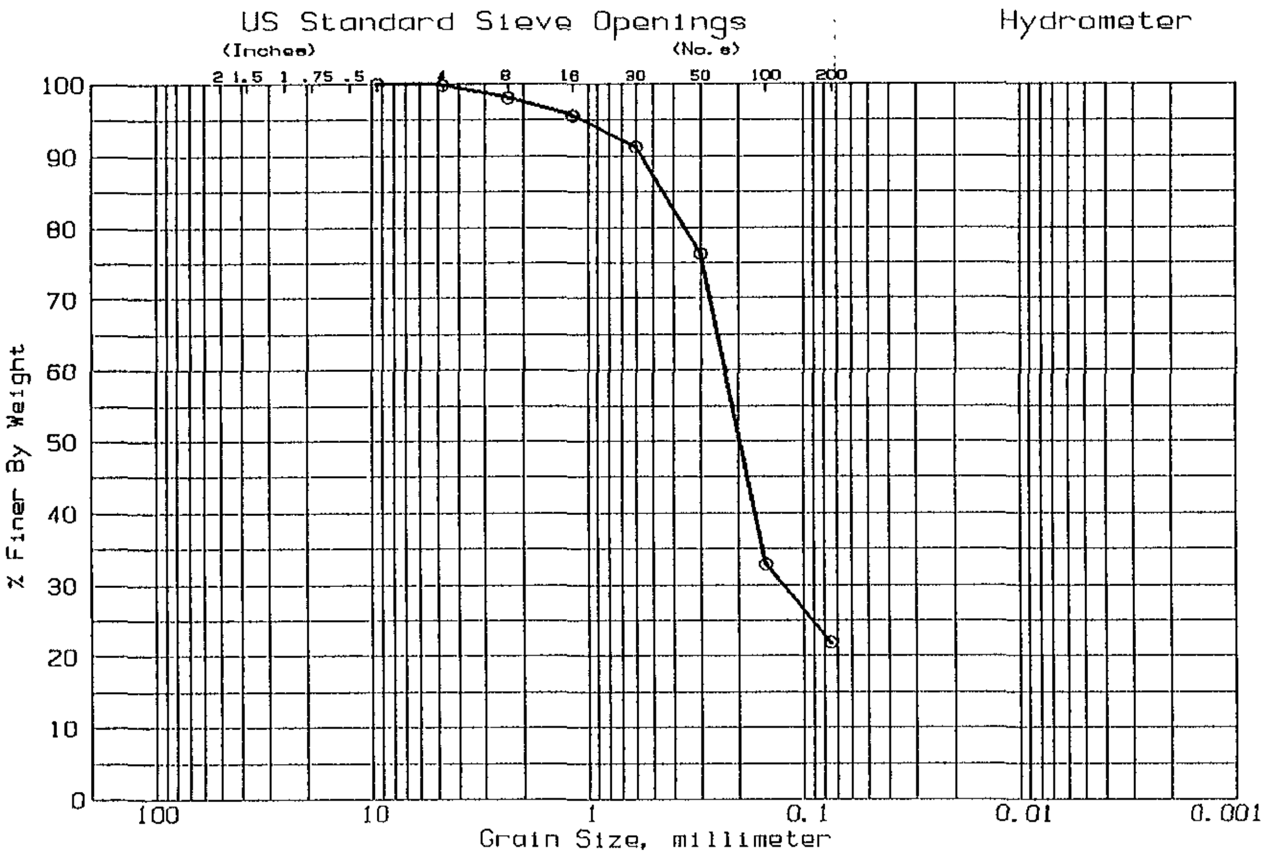


CONSOLIDATION — PRESSURE TEST DATA

BSK & Associates

OUR JOB P91083
June 1991
Figure 10

PARTICLE SIZE ANALYSIS



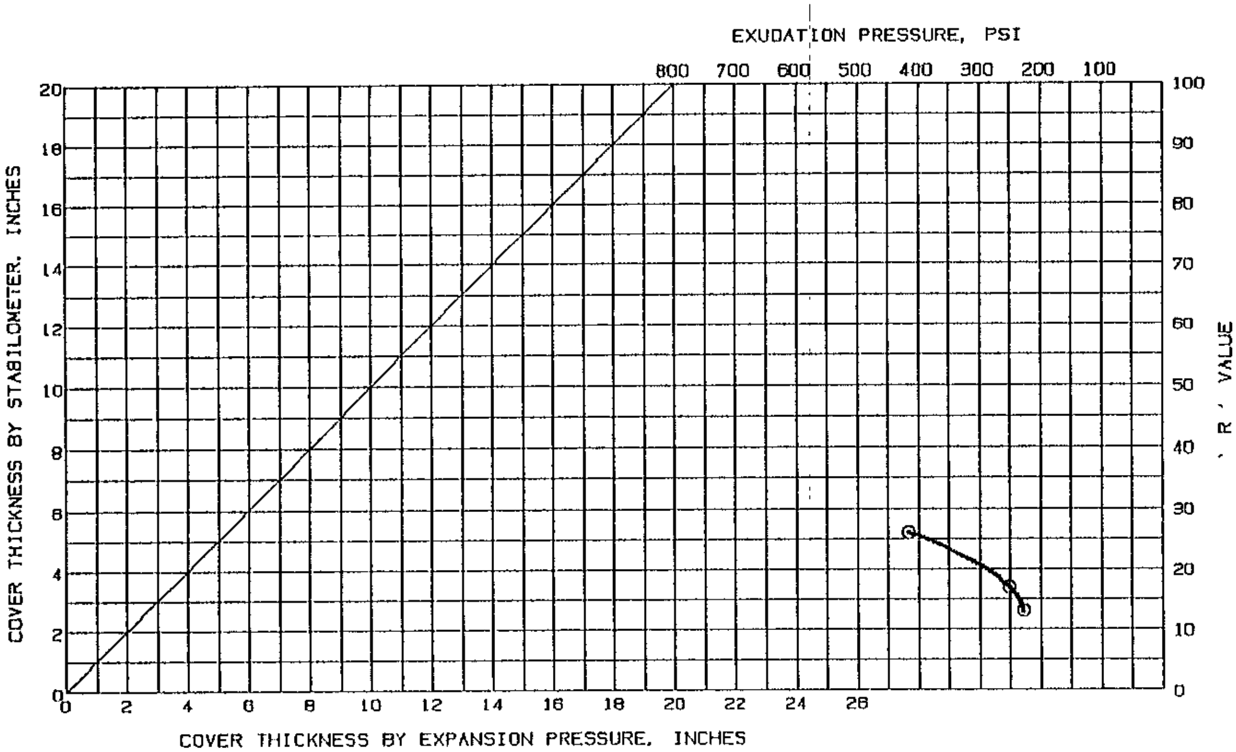
Cobble	Gravel		Sand			Silt	Clay
	Coarse	Fine	Coarse	Medium	Fine		

Unified Soil Classification System (ASTM D2487)

Symbol	o
Boring No.	B-1
Sample Depth	36.5 to 38 feet
Percent Gravel	0
Percent Sand	79
Percent Silt & Clay	21

OUR JOB P91083
June 1991
Figure 11

RESISTANCE VALUE TEST



Designation: 1

Location: B-2; B-3; & B-4; Composite; 0 to 3 feet

Description: Dark Brown to Black SILTY CLAY

TEST DATA			
Specimen	A	B	C
Moisture Content at Test, %	13.1	14.5	15.4
Dry Density at Test, pcf	120	116	114
Exudation Pressure, psi	416	252	228
Expansion Dial (.0001 in.)	0	0	0
Resistance Value, ' R '	26	17	13

' R ' Value at 300 psi Exudation Pressure: 21

BSK & Associates