GEOTECHNICAL INVESTIGATION REPORT
PROPOSED TELECOMMUNICATIONS FACILITY
HANK'S TOWING, SITE NO. FN03XC014
16065 MATEO STREET
SAN LEANDRO, CALIFORNIA

INTRODUCTION

GENERAL

This report presents the results of our geotechnical investigation for a proposed telecommunications facility to be located at 16065 Mateo Street in San Leandro, California. The purpose of our investigation was to explore and evaluate the subsurface conditions at the site in order to develop recommendations related to the geotechnical aspects of project design and construction.

The site location relative to existing roads and topographic features is shown on Plate 1.

PROPOSED CONSTRUCTION

We understand the proposed project will involve construction of a telecommunications facility on a rectangularly-shaped site. The planned facility will include the installation of a steel monopole telecommunications tower, approximately 50 feet in height, and adjacent equipment cabinets. Appurtenant construction may include underground utilities.

Plans indicating final site grades were not available at the time this report was prepared; however, as existing site topography is relatively level (and the surface paved), we anticipate little-to-no earthwork grading will be performed for this project. Excavations for underground utilities are not anticipated to exceed about 5 feet below existing or final site grades.

A plot plan indicating the proposed project area is presented on Plate 2.

SCOPE OF SERVICES

Our services for this project were performed in general accordance with Sprint's Attachment A-3, "Drilling, Testing, and Geotechnical Requirements," and included the following:

- Review readily available literature pertaining to site geology, faulting, and seismicity.
- Exploration of the subsurface conditions at the site using one boring excavated with a truck-mounted drill rig.
- Preparation of this report which includes:
  - A description of the proposed project;
  - A summary of our field exploration program;
  - A discussion of site geology, faulting, and seismicity based on our review of readily available geologic literature;
  - A description of site surface and subsurface conditions encountered during our field investigation;
  - Our comments regarding potential geologic hazards which could affect the site or proposed project; and
  - Recommendations related to the geotechnical aspects of site preparation and engineered fill, temporary excavations and trench backfill, and foundation design and construction.

FIELD INVESTIGATION

Subsurface conditions at the site were explored on September 11, 2000, by drilling one boring (designated B-1) to a depth of about 26-1/2 feet below existing site grade. The boring was advanced using a CME 55, truck-mounted drill rig equipped with a 6-inch-diameter, hollow-stem auger. The approximate location of the exploratory boring performed for this investigation is indicated on Plate 2.

Our geologist maintained a log of the boring, visually classified the soils encountered according to the Unified Soil Classification System (see Plate 3), and obtained relatively undisturbed samples of the subsurface materials. Soil samples were obtained from the boring with a Standard Penetration

Sampler driven 18 inches (unless otherwise noted) into undisturbed earth materials using a 140-pound hammer falling 30 inches. After the boring was completed, it was backfilled with the drill cuttings. A log of the exploratory boring performed for this investigation is presented on Plate 4.

SITE CONDITIONS

GEOLOGY AND SEISMICITY

Geologic Setting

The project site is located within the Coast Range geologic province. The geologic structure of this province is complex, having been molded by numerous mountain building events characterized by extensive folding, faulting, and fracturing of variable intensity. Regionally, these folds and faults trend northwesterly and are responsible for the development of a pronounced northwest trending ridge-valley system.

Based on our review of the California Division of Mines and Geology map entitled: "Geologic Map of the San Francisco-San Jose Quadrangle, California," compiled by D.L. Wagner, E.J. Bortugno, and R.D. McJunkin (published 1991), the project site lies within an area of Quaternary-age alluvium.

Faulting and Seismicity

The closest, active[1] fault mapped by the California Division of Mines and Geology[2] is the Hayward Fault Zone, located on the order of 1,100 feet to the northeast of the site. Other major active faults (or fault zones) within the immediate site vicinity include the Calaveras Fault (located approximately 9 miles to the east), the Pleasanton Fault (located approximately 11 miles to the east), and the Concord Fault (located approximately 17 miles to the northeast).

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[1] Within this report, a fault is considered active if there is evidence of Holocene (i.e., within the past 10,000 to 12,000 years) surface displacement along one or more of its segments or branches.

[2] Reference: California Division of Mines and Geology map entitled: "Map Showing Recency of Faulting, San Francisco-San Jose Quadrangle, California," compiled by D.L. Wagner, E.J. Bortugno, and R.D. McJunkin (published 1991).

SURFACE

The project site consists of a rectangularly-shaped area located at 16065 Mateo Street in San Leandro, California. The site is bounded to the northeast by gravel-surfaced and asphalt-concrete-paved parking/driveway areas (with existing residences beyond), to the southeast by gravel-surfaced parking areas and numerous stored vehicles (with a church facility beyond), to the southwest by a fence (with commercial/industrial development beyond), and to the northwest by a garage-type structure (with asphalt-concrete-paved parking/driveway areas beyond). At the time of our field investigation, the site area was surfaced with asphalt-concrete and appeared to be used for vehicular access. Additionally, utility boxes and a propane tank were present within the southwest portion of the site. Existing topography within the immediate site area was relatively level.

SUBSURFACE

Near-surface earth materials encountered in the boring performed for this investigation (and beneath on-site pavements) consisted of fill composed predominantly of medium dense silty gravel to a depth of about ½-foot below existing site grade. Based on our observations of the site area, we suspect encountered fill represents surfacing materials for vehicular parking/driveway areas. Below these near-surface fill soils, medium-stiff-to-very-stiff sandy clay, loose silty sand, stiff-to-very-stiff silty clay, loose-to-medium-dense silty sand/sandy silt, and stiff clayey silt were encountered to the maximum depth explored (approximately 26-1/2 feet below existing site grade).

Free groundwater was encountered during our field investigation at a depth of about 8-1/2 feet below existing site grade. However, groundwater conditions can vary depending on the season, precipitation, runoff conditions, irrigation and/or groundwater pumping practices (both on and off site), the level of nearby bodies of water, and possibly other factors. Therefore, groundwater conditions presented in this report may not be representative of those which may be encountered during or subsequent to construction.

A more detailed description of the subsurface conditions encountered during our field investigation is provided on the attached log.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

Based on the results of our investigation, it is our professional opinion the proposed steel monopole telecommunications tower may be supported using a drilled, cast-in-place concrete pier; foundation support for the proposed equipment cabinets may be provided using a mat foundation (or foundations).

Though we anticipate the planned facility may be designed and constructed generally using conventional foundations, it should be noted existing fill, potentially expansive clay soils, shallow groundwater, and cohesionless sandy soils were encountered during our field exploration program. Based on the scope of the currently proposed project, the presence of existing fill and potentially expansive clay soils should not have a significant adverse effect on project features. However, if the nature of the proposed construction changes (i.e., buildings, pavements, or other improvements sensitive to ground movement and/or settlement are to be constructed at the site), special design and construction provisions may be required.

The presence of groundwater (initially encountered at a depth of approximately 8-1/2 feet below existing site grade) and cohesionless sandy soils (encountered at depths of about 7 to 9 and 19 to 24 feet below existing site grade) will likely hinder the drilling operation for the proposed tower foundation pier, requiring casing, drilling fluids, and/or other methods to advance the excavation and maintain hole stability.

Specific comments regarding the conditions outlined above, as well as recommendations regarding the geotechnical aspects of project design and construction, are presented in the following sections of this report.

GEOLOGIC HAZARDS

Ground Shaking

No active faults are known to cross the site area, nor is the site within an Alquist-Priolo Zone. The closest mapped, significant active fault (or fault zone) is the Hayward Fault Zone, located on the order of 1,100 feet northeast of the site. Based on the distance to this nearest fault, it is our professional opinion that the potential for ground rupture (or other similar effect) at the site in the event of a seismic event is highly unlikely. However, the site could be subjected to strong ground shaking in the event of an earthquake on any one of several, nearby faults. Therefore, we recommend the planned steel monopole telecommunications tower, equipment cabinets, and any other site improvements sensitive to ground shaking be designed for a strong level of ground shaking (i.e., a peak horizontal ground acceleration on the order of 0.7 to 0.8g[3], where "g" equals 32.2 feet per second per second).

In the event the Uniform Building Code (UBC) is used for seismic design, we would recommend structural features of the project be designed using a Type S_E soil profile and Near-Source Factors, N_a and N_v, of 1.5 and 2.0, respectively. These soils- and fault-related parameters are based on the results of our field investigation, our general knowledge of subsurface conditions within the site area, and our review of current fault information (i.e., the California Department of Conservation, Division of Mines and Geology publication entitled: "Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada," published by the International Conference of Building Officials, February, 1998).

Liquefaction

Liquefaction is a phenomenon whereby loose, saturated, granular soil deposits lose a significant portion of their shear strength due to excess pore water pressure buildup resulting from cyclic loading, such as that caused by an earthquake. Among other effects, liquefaction can result in densification of such deposits after an earthquake as excess pore pressures are dissipated (and hence settlements of overlying deposits). The primary factors deciding liquefaction potential of a soil deposit are: (1) the level and duration of seismic ground motions; (2) the type and consistency of the soils; and (3) the depth to groundwater.

Subsurface earth materials encountered during our field investigation generally consisted of medium dense silty gravel underlain by medium-stiff-to-very-stiff sandy clay, loose silty sand, stiff-to-very-stiff silty clay, loose-to-medium-dense silty sand/sandy silt, and stiff clayey silt. Free groundwater was encountered during our field investigation at a depth of about 8-1/2 feet below existing site grade.

Based on the generally fine-grained and/or relatively stiff nature of the soils encountered during our field investigation, it is our professional opinion the potential for liquefaction at the site during or subsequent to a seismic event is unlikely.

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[3] Reference: "Seismic Shaking Hazard Maps of California," California Department of Conservation, Division of Mines and Geology; Map Sheet 48, 1999.

Ground Subsidence

Ground subsidence within the site area would typically be due to densification of subsurface soils during or subsequent to a seismic event. Generally, loose, granular soils would be most susceptible to densification, resulting in ground subsidence.

Given the generally fine-grained and/or relatively stiff nature of the soils encountered during our field investigation, it is our professional opinion that the potential for significant ground subsidence at the site during or subsequent to a seismic event is unlikely.

Landslides

The site of the proposed telecommunications facility is in an area of relatively level topography. Since little-to-no earthwork grading is anticipated for the project, it is our professional opinion that landsliding is unlikely at the site and that earthwork grading (if any) should not result in a potential for slope instability within or in the immediate vicinity of the site.

EXPANSIVE SOIL

Based on the results of our field exploration program, near-surface clay soils located at the site appear to be expansive. Expansive soils are characterized by their ability to undergo significant volume change (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from rainfall, landscape irrigation, utility leakage, roof drainage, drought, or other factors, and may cause unacceptable settlement or heave of structures, concrete slabs supported-on-grade, or pavements supported over these materials.

In our opinion (and based on the scope of the currently proposed project), the presence of potentially expansive surficial clay soils should not have a significant adverse effect on currently-planned project features. However, if the nature of the proposed construction changes (i.e., buildings, pavements, or other improvements sensitive to ground movement are to be constructed at the site), special design and construction provisions may be required. Such provisions could include moisture conditioning slab and pavement subgrade soils and strengthening foundations and slabs. In the event the nature of the proposed construction changes, we should be notified immediately in order to review and, if deemed necessary, conduct additional studies and/or provide supplemental recommendations.

EXISTING, ON-SITE FILL

Based on the results of our field investigation and site observations, it appears existing fill is present within the site area to a depth of at least ½-foot below existing site grade. In our opinion (and based on the scope of the currently proposed project), the presence of this fill should not have a significant adverse effect on planned project features. However, if the nature of the proposed construction changes (i.e., buildings, pavements, or other improvements sensitive to settlement are to be constructed at the site), special design and construction provisions may be required. Such provisions could include removal of on-site fill and replacement with engineered fill, or deepening structural foundations through these materials.

SITE PREPARATION

Removal of Existing Pavements

We anticipate existing asphalt concrete pavements located within the planned construction area will be demolished during initial site development. In general, we would recommend these materials be disposed of off-site or outside the construction limits.

Existing Utilities

If abandoned, below-grade utility lines are encountered within the area of construction they should be removed and disposed of off-site. Existing, below-grade utility pipelines (if any) which extend beyond the limits of the proposed construction and will be abandoned in-place should be plugged with cement grout to prevent migration of soil and/or water. All excavations resulting from removal activities should be cleaned of all loose or disturbed material (including previously-placed backfill) prior to placing any fill or backfill.

Wet/Unstable Soil Conditions

It has been our experience that soils located directly beneath existing pavements can be significantly over optimum moisture content. This condition could hinder equipment as well as efforts to compact site soils to a specified level of compaction. Disking to aerate, replacement with imported material, chemical treatment, stabilization with a geotextile fabric or grid, and/or other methods will likely be required to facilitate earthwork operations (if any). The applicable method will depend on the contractor's capabilities as well as other project-related factors beyond the scope of this study. Therefore, if over-optimum soil conditions are encountered during construction, the project Geotechnical Engineer should review these conditions (as well as the contractor's capabilities) and, if appropriate, provide recommendations for their treatment.

TEMPORARY EXCAVATIONS

General

All excavations must comply with applicable local, state, and federal safety regulations including the current OSHA Excavation and Trench Safety Standards. Construction site safety generally is the responsibility of the contractor, who should be solely responsible for the means, methods, and sequencing of construction operations.

Construction Considerations

Heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within 5 feet of the top of any excavation. Where the stability of adjoining buildings, walls, or other structures is endangered by excavation operations, support systems such as shoring, bracing, or underpinning may be required to provide structural stability and to protect personnel working within the excavation.

During wet weather, earthen berms or other methods should be used to prevent runoff water from entering all excavations. All runoff water entering the excavation(s) should be collected and disposed of outside the construction limits.

Excavation Conditions

Based on our experience in the site area and conditions encountered during our field exploration program, we anticipate trench (and other shallow) excavations should be possible with a conventional backhoe (such as a Case 580 or equivalent).

TRENCH BACKFILL

Materials

Pipe zone backfill (i.e., material beneath and in the immediate vicinity of the pipe) should consist of on-site or imported soil and/or soil-aggregate mixtures generally less than 1 inch in maximum dimension and free of organic or other deleterious debris; trench zone backfill (i.e., material placed between the pipe zone backfill and finished subgrade) may consist of on-site soil, generally less than 3 inches in maximum and free of organic or other deleterious debris.

If imported material is used for pipe or trench zone backfill, we recommend it consist of fine-grained sand. In general, use of coarse-grained sand and/or gravel is not recommended due to the potential for soil migration into, and water seepage along, trenches backfilled with this type of material.

Recommendations provided above for pipe zone backfill are minimum requirements only. More stringent material specifications may be required to fulfill local codes and/or bedding requirements for specific types of pipe. We recommend the project Civil Engineer develop these material specifications based on planned pipe types, bedding conditions, and other factors beyond the scope of this study.

Placement and Compaction

Trench backfill consisting of on-site soils should be uniformly moisture-conditioned to at least 3 (but generally no more than about 5) percent above the optimum moisture content, placed in horizontal lifts less than 8 inches in loose thickness, and compacted to at least 88 (but generally no more than about 92) percent of the maximum dry density as determined by ASTM (American Society for Testing and Materials) Test Method D 1557[4]. Trench backfill consisting of imported soils (or soil-aggregate mixtures) should be uniformly moisture-conditioned to between 0 and 5 percent above the optimum moisture content, placed in horizontal lifts less than 8 inches in loose thickness, and compacted to at least 90 percent relative compaction. Within pavement areas, all trench backfill should be compacted to at least 95 percent relative compaction within 12 inches of finished subgrade[5]. Mechanical compaction is strongly recommended; ponding or jetting should not be allowed unless specifically reviewed and approved by the project Geotechnical Engineer prior to construction.

ENGINEERED FILL

Materials

As site topography within the area of planned improvements is relatively level (and the site is paved), we anticipate little-to-no earthwork grading will be performed for this project. However, some fill may be required to backfill around foundations or for other purposes. If required, we recommend this material consist of on-site or imported[6] soil and/or soil-aggregate mixtures generally less than

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[4] This test procedure should be used wherever relative compaction, maximum dry density, or optimum moisture content is referenced within this report.

[5] Within this report, finished subgrade refers to the top surface of undisturbed on-site soil compacted during site preparation, compacted trench backfill, and/or engineered fill.

[6] All imported soil and/or soil-aggregate mixtures used for engineered fill should be sampled, tested and approved by the project Geotechnical Engineer prior to being transported to the site.

3 inches in maximum dimension, nearly-free of organic or other deleterious debris, and essentially non-plastic. Typically, well-graded mixtures of gravel, sand, non-plastic silt, and small quantities of clay would be acceptable for use as engineered fill.

Placement and Compaction

On-site soils used for engineered fill should be uniformly moisture-conditioned to at least 3 (but generally no more than about 5) percent above the optimum moisture content, placed in horizontal lifts less than 8 inches in loose thickness, and compacted to at least 88 (but generally no more than about 92) percent relative compaction. Within pavement areas, these soils should be uniformly moisture-conditioned to at least 1 (but generally no more than about 3) percent above optimum and compacted to at least 95 percent relative compaction within 12 inches of finished subgrade.

Imported soils (or soil-aggregate mixtures) used for engineered fill should be uniformly moisture-conditioned to between 0 and 5 percent above the optimum moisture content, placed in horizontal lifts less than 8 inches in loose thickness, and compacted to at least 90 percent relative compaction. In pavement areas, engineered fill placed within 12 inches of finished subgrade should be compacted to at least 95 percent relative compaction.

TOWER FOUNDATION - DRILLED PIER

General

We anticipate the planned monopole telecommunications tower will be subject to relatively high lateral foundation loads. Typically, such loads are resisted using a deep foundation (i.e., a drilled pier). Based on the results of our investigation, it is our professional opinion a drilled, cast-in-place concrete pier may be used for support of the planned tower. In general, this pier should consist of a drilled, straight-shafted hole, filled with concrete, and reinforced with steel to resist and transfer lateral and axial loads. Further, we recommend the proposed pier extend to a depth of at least 15 feet below existing (and final) adjacent site grades, have a diameter of at least 2 feet, and generally not extend below a depth of about 26-1/2 feet below existing site grade (the approximate maximum depth explored during this investigation).

Design parameters as well as construction recommendations for a drilled, cast-in-place concrete pier are provided on the following pages.

Axial Capacities

A cast-in-place concrete pier constructed in accordance with recommendations provided herein may be designed to resist downward loads using an allowable end bearing pressure of 1,000 pounds per square foot (psf) and a unit skin friction of 150 psf. The uppermost 3 feet of the embedded portion of the pier should be neglected when evaluating the skin friction component of the axial capacities.

The allowable end bearing pressure provided above is a net value; therefore, the weight of the pier may be neglected when evaluating downward capacities. Total downward capacities derived from the parameters provided above may be increased by 1/3 for short-term loading due to wind or seismic forces.

Estimated Settlement

Total settlement of the proposed pier is estimated to be less than 3/4-inch and should occur shortly after the initial loads are applied.

Lateral Capacities

We recommend lateral resistance and deflection of the proposed pier be evaluated using methods proposed by Broms for rigid piles[7]. For this method, we recommend a coefficient of horizontal subgrade reaction of 5 tons per cubic foot and a cohesive strength of 400 pounds per square foot be used to evaluate lateral capacities and deflections.

Since the aforementioned method requires information regarding the proposed pier (i.e., depth of embedment, pier diameter, pier length, lateral loads, and location of loading) which was not available at the time this report was prepared, we recommend the project Structural Engineer evaluate lateral deflections of the proposed pier (if required).

Alternatively, lateral capacity may be evaluated using the "Pole Formula" given in Sections 1806.8.2.1 and 1806.8.2.2 of the Uniform Building Code (UBC, Volume 2, 1997 edition). For this method we recommend a lateral soil bearing pressure of 100 pounds per square foot per foot of embedment be used for analysis. If applicable, the 100 percent increase allowed by the Code for isolated poles (which are not adversely affected by a ½-inch horizontal deflection at the ground surface due to short-term lateral loads) may be used for design.

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[7] Broms, Bengt B., "Lateral Resistance of Piles in Cohesive Soils," *Journal of Soil Mechanics and Foundations Division*, ASCE, March, 1964.

To account for possible loss of subgrade support due to surface disturbance, we recommend soil located within the uppermost two feet of the embedded portion of the pier be neglected when evaluating lateral capacities and/or deflections.

Excavation Conditions

Based on the conditions encountered during our field exploration program, we anticipate excavations for the proposed pier should be possible using a large, truck-mounted drill rig equipped with a hydraulically-advanced, bucket auger. However, due to the presence of groundwater and cohesionless soils, drilled excavations for the proposed tower foundation pier will likely be susceptible to caving, especially between depths of about 7 to 9 and 19 to 24 feet below existing site grade. In our opinion, casing, drilling fluids, and/or other methods will likely be required to advance the excavation and maintain hole stability.

Casing

If casing is used, we recommend it be removed from the excavation as concrete is being placed. The bottom of the casing should be maintained below the top of the concrete at all times during casing withdrawal and concrete placement. Further, continuous vibration or other approved methods should be used during casing withdrawal to reduce the potential for void-space formation within the concrete. Abandoning the casing in-place should not be allowed.

Drilling Fluids

If drilling fluids[8] are used to facilitate construction of the proposed drilled pier, we recommend steel reinforcement and concrete be placed immediately upon completion of the pier excavation to reduce the quantity of suspended soil particles which may settle to the bottom of the hole. Further, we recommend all pier construction operations which utilize drilling fluids be in accordance with procedures outlined in the Federal Highway Administration publication entitled: *Drilled Shafts: Construction Procedures and Design Methods*.

Bottom Preparation

All debris and any loose or disturbed soil should be removed from the pier excavation just prior to placing reinforcing steel and/or concrete. A representative from Brown & Mills should observe the pier excavation to verify that subsurface conditions are consistent with those encountered during our field investigation.

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[8] Drilling fluids are typically composed of water mixed with bentonite or a synthetic thickener to increase density and consistency.

Steel and Concrete Placement

Reinforcing steel and/or concrete should be placed immediately upon completion of the pier excavation. If water is present during concrete placement, concrete should be pumped or otherwise discharged to the bottom of the hole via a hose or tremie pipe. The end of the hose or tremie pipe must remain below the top surface of any water and/or the in-place concrete at all times. Additionally, concrete (used for pier construction) should be consolidated using vibratory methods upon removal of water and/or drilling fluids.

In order to develop the design skin friction value provided above, concrete used for pier construction should have a slump of from 4 to 6 inches if placed in a dry shaft without temporary casing, and from 6 to 8 inches if casing and/or drilling fluids are used. The concrete mix should be designed with appropriate admixtures and/or water/cement ratios to achieve these recommended slumps. Adding water to a conventional mix to achieve the recommended slump should not be allowed.

EQUIPMENT CABINET FOUNDATIONS

General

Foundation support for planned equipment cabinets may be provided using a mat foundation (or foundations). All proposed mat foundations should be constructed of reinforced concrete, a minimum of 18 inches wide, embedded a minimum of 12 inches below the lowest adjacent final subgrade[9], and founded on undisturbed on-site soil and/or engineered fill.

Allowable Bearing Pressure

An allowable bearing pressure of 1,500 pounds per square foot (psf) may be used for the design of proposed equipments mat foundations which possess the above minimum dimensions. The allowable bearing pressure provided is a net value; therefore, the weight of the foundation (which extends below finished subgrade) may be neglected when computing dead loads. The allowable bearing pressure provided herein applies to dead plus live loads, includes a calculated factor of safety of at least 3, and may be increased by 1/3 for short-term loading due to wind or seismic forces. For mat foundations subject to overturning forces, the maximum edge pressure should not exceed the allowable bearing pressure.

[9] Within this report, final subgrade refers to the top surface of undisturbed on-site soil, on-site soil compacted during site preparation, and/or engineered fill.

Lateral Resistance

Resistance to lateral loads (including those due to wind or seismic forces) may be provided by frictional resistance between the bottom of proposed concrete mat foundations and the underlying soil, and by passive earth pressure against the sides of the foundations. A coefficient of friction of 0.3 may be used between cast-in-place concrete foundations and the underlying soil; passive pressure available in undisturbed on-site soil and/or engineered fill may be taken as equivalent to the pressure exerted by a fluid weighing 275 pounds per cubic foot (pcf). To account for possible future loss of subgrade support due to surface disturbance, we recommend earth materials located within the uppermost 1/2-foot of the embedded portion of all shallow foundations be neglected when evaluating passive pressures.

Lateral resistance parameters provided above are ultimate values. Therefore, a suitable factor of safety should be applied to these values for design purposes. The appropriate factor of safety will depend on the design condition and should be determined by the project Structural Engineer. Depending on the application, typical factors of safety could range from 1.0 to 1.5.

Expansive Soil Considerations

Shallowly-embedded mat foundations (i.e., embedded less than 2 to 3 feet below finished grade) may be susceptible to minor vertical movements (i.e., less than 1-inch) due to shrinking or swelling of the subgrade soils. In general, we would not anticipate such movements to have a significant adverse effect on planned equipment supported on these mats. However, minor vertical movements may adversely affect below-grade utility connections between the proposed mat (or mats) and below-grade utilities (if any). Should such utilities exist, we recommend connections be designed to resist (or accept) a 1-inch vertical movement, either up or down.

Construction Considerations

Prior to placing steel or concrete, foundation excavations should be cleaned of all debris, loose or disturbed soil, and any water.

ADDITIONAL SERVICES

We recommend Brown & Mills review final earthwork grading (if any) and/or foundation plans and specifications to evaluate that recommendations contained herein have been properly interpreted and implemented during design. Further, all site earthwork activities, including site preparation, placement of engineered fill and trench backfill, and all foundation excavations (including those for the proposed tower foundation pier) should be monitored by a representative from Brown & Mills.

Monitoring services are an essential component of our design services. Monitoring allows us to observe the soil conditions encountered during construction, evaluate the applicability of the recommendations presented in this report to the soil conditions encountered, and recommend appropriate changes in design or construction procedures if conditions differ from those described herein.

LIMITATIONS

This report has been prepared in substantial accordance with the generally accepted geotechnical engineering practice as it existed in the site area at the time our services were rendered. No warranty is either expressed or implied.

Conclusions and recommendations contained in this report were based on the conditions encountered during our field investigation and are applicable only to those project features described above (see section entitled "PROPOSED CONSTRUCTION"). It is possible subsurface conditions could vary beyond the point explored. If conditions are encountered during construction which differ from those described in this report, or if the scope or nature of the proposed construction changes, we should be notified immediately in order to review and, if deemed necessary, conduct additional studies and/or provide supplemental recommendations.

Recommendations provided in this report are based on the assumption that an adequate program of tests and observations will be conducted by Brown & Mills during the construction phase in order to evaluate compliance with our recommendations.

The scope of services provided by Brown & Mills for this project did not include the investigation and/or evaluation of toxic substances, or soil or groundwater contamination of any type. If such conditions are encountered during site development, additional studies may be required. Further, services provided by Brown & Mills for this project did not include the investigation and/or evaluation of soil corrosivity. Depending on planned pipe types, bedding conditions, and other factors beyond the scope of this study, it may be appropriate to evaluate soil corrosivity prior to development.

This report may be used only by our client, and only for the purposes stated herein, within a reasonable time from its issuance. Land use, site conditions, and other factors may change over time which may require additional studies. In the event a significant period of time elapses between the date of this report and construction, Brown & Mills shall be notified of such occurrence in order to review current conditions. Depending on that review, additional studies and/or an updated or revised report may be required prior to completion of final design.

Any party other than our client who wishes to use all or any portion of this report shall notify Brown & Mills of such intended use. Based on the intended use as well as other site-related factors, Brown & Mills may require that additional studies be conducted and that an updated or revised report be issued. Failure to comply with any of the requirements outlined above by the client or any other party shall release Brown & Mills from any liability arising from the unauthorized use of this report.

NORTH

SCALE 1:62,500

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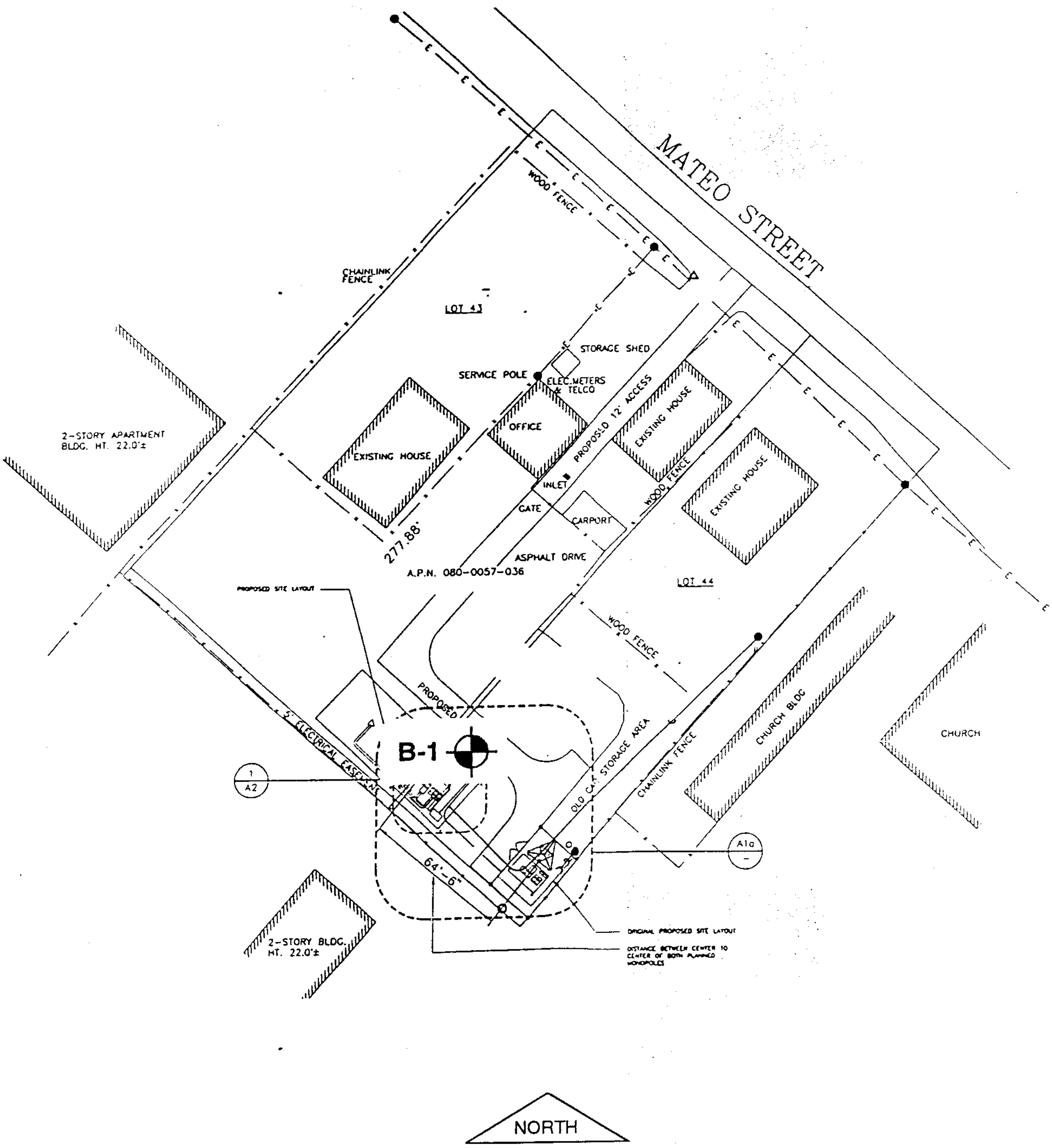
BMI

BMI PROJECT NO. ▶ 00S-753

VICINITY MAP
PROPOSED TELECOMMUNICATIONS FACILITY
HANKS TOWING, SITE NO. FN03XC014
SAN LEANDRO, CALIFORNIA

PLATE

1



NO SCALE

LEGEND

APPROXIMATE BORING LOCATION

NOTE: The boring was located in the field by visual methods. Therefore, the location of the boring shown on this plan should be considered highly approximate.

REFERENCE: Plan prepared by Delta Groups Engineering, Inc. titled "SITE PLAN", dated 10/28/99 (latest revision).

BMI	**SITE PLAN** **PROPOSED TELECOMMUNICATIONS FACILITY** **HANKS TOWING, SITE NO. FN03XC014** **SAN LEANDRO, CALIFORNIA**	PLATE **2**
BMI PROJECT NO. ▶ 00S-753		

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			SYM.	DESCRIPTION
COARSE-GRAINED SOILS MORE THAN 50% OF MATERIAL IS GREATER THAN NO. 200 SIEVE	GRAVELS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS (LITTLE OR NO FINES)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
		GRAVELS (APPRECIABLE FINES)	GM	Silty gravels, poorly-graded gravel-sand-silt mixtures
			GC	Clayey gravels, poorly-graded gravel-sand-clay mixtures
	SANDS MORE THAN 50% OF COARSE FRACTION PASSES NO. 4 SIEVE	SANDS (LITTLE OR NO FINES)	SW	Well-graded sands, gravelly sands, little or no fines
			SP	Poorly-graded sands, gravelly sands, little or no fines
		SANDS (APPRECIABLE FINES)	SM	Silty sands, poorly-graded sand-gravel-silt mixtures
			SC	Clayey sands, poorly-graded sand-gravel-clay mixtures
FINE-GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	Inorganic silts and very fine sands, silty or clayey fine sands, 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 clays of low plasticity
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic silts and clays of high plasticity
HIGHLY ORGANIC SOILS			PT	Peat, humus, swamp soils with high organic content

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

LOG SYMBOLS AND DEFINITIONS

FIELD

- STANDARD PENETRATION SPLIT-SPOON SAMPLER (2-INCH OUTSIDE DIAMETER)
- CALIFORNIA SAMPLER (3-INCH OUTSIDE DIAMETER)
- MODIFIED CALIFORNIA SAMPLER (2.5-INCH OUTSIDE DIAMETER)
- BAG/BULK
- THIN-WALLED SHELBY TUBE (3-INCH OUTSIDE DIAMETER)
- WATER LEVEL (LEVEL ESTABLISHED AS NOTED ON LOGS)
- WATER OR SEEPAGE ENCOUNTERED (LEVEL NOT ESTABLISHED)

LABORATORY

-4	% PASSING NO. 4 SIEVE (ASTM TEST METHOD C 136)
-200	% PASSING NO. 200 SIEVE (ASTM TEST METHOD C 117)
LL	LIQUID LIMIT (ASTM TEST METHOD D 4318)
PI	PLASTICITY INDEX (ASTM TEST METHOD D 4318)
R-VAL	RESISTANCE VALUE (CALTRANS TEST 301)
EI	EXPANSION INDEX (UBC STANDARD 29-2)
COL	COLLAPSE POTENTIAL (ASTM TEST METHOD D 5333)
SP	SWELL POTENTIAL (under a specified load) (ASTM TEST METHOD D 4546)
SL	SWELL PRESSURE (no consolidation) (ASTM TEST METHOD D 4546)

GENERAL NOTES:
1. Lines separating soil or rock strata on logs are approximate boundaries only. Actual transitions may be gradual and, in the case of selectively sampled borings, may vary by as much as the sample interval.
2. In general, Unified Soil Classification designations were evaluated using visual methods only. Actual designations (based on laboratory tests) may vary.
3. Logs represent general soil conditions on the date and at the location indicated. No warranty is provided as to the continuity of soil conditions between individual sample locations.
4. Unconfined compressive strengths reported on the logs (if any) were obtained using a pocket penetrometer.

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LOG LEGEND
PROPOSED TELECOMMUNICATIONS FACILITY
HANKS TOWING, SITE NO. FN03XC014
SAN LEANDRO, CALIFORNIA

PLATE
3

EXPLORATION DATE	LOGGED BY	TOTAL DEPTH	BORING NO.
September 11, 2000	Brian Richardson	26-1/2 feet	**B-1**
EXPLORATION EQUIPMENT		BACKFILL MATERIAL	
CME 55 equipped with a 6-inch-diameter, hollow-stem auger		Drill cuttings	

SURFACE CONDITIONS

Relatively level, gravel-surfaced driveway and parking

GROUNDWATER CONDITIONS

Free groundwater encountered at a depth of approximately 8-1/2 feet below existing site grade.

APPROX. GROUND SURFACE ELEVATION (IN FEET) ▶ N/A

FIELD						DESCRIPTION	LABORATORY		
DEPTH (IN FEET)	SAMPLE TYPE	SAMPLE NO.	BLOWS/FOOT	UNCONFINED COMP. STRENGTH (TSF)	USCS LETTER SYMBOL		DRY DENSITY (PCF)	MOISTURE CONTENT (%)	OTHER LAB TESTS SEE LOG LEGEND FOR ABBREVIATION DEFINITIONS
		1			GM	***Silty GRAVEL:*** Mottled orange-brown and dark brown, dry, medium dense, fine grained, with some medium-to-coarse sand and clay (FILL)			
		2	16	2.0	CL	***Sandy CLAY:*** Dark brown, moist, stiff-to-very-stiff, fine grained, with some silt			
5		3	6	0.5		grades medium stiff, with more fine sand			
					SM	***Silty SAND:*** Olive-brown, moist, loose, fine-to-medium grained			
10		4	8	1.5	CL	***Silty CLAY:*** Dark brown, moist, stiff			
15		5	14	2.0		grades mottled orange-brown and light brown, stiff-to-very stiff			
20		6	10		SM/ML	***Silty SAND/Sandy SILT:*** Olive-brown to yellow-brown, wet, loose-to-medium-dense, fine grained, with trace clay grades with more clay and less sand			
25		7	7	1.0	ML	***Clayey SILT:*** Yellow-brown, moist-to-wet, stiff, with some fine sand			
30									

BMI PROJECT NO. ▶ 00S-753

LOG OF EXPLORATORY BORING
PROPOSED TELECOMMUNICATIONS FACILITY
HANKS TOWING, SITE NO. FN03XC014
SAN LEANDRO, CALIFORNIA

PLATE 4