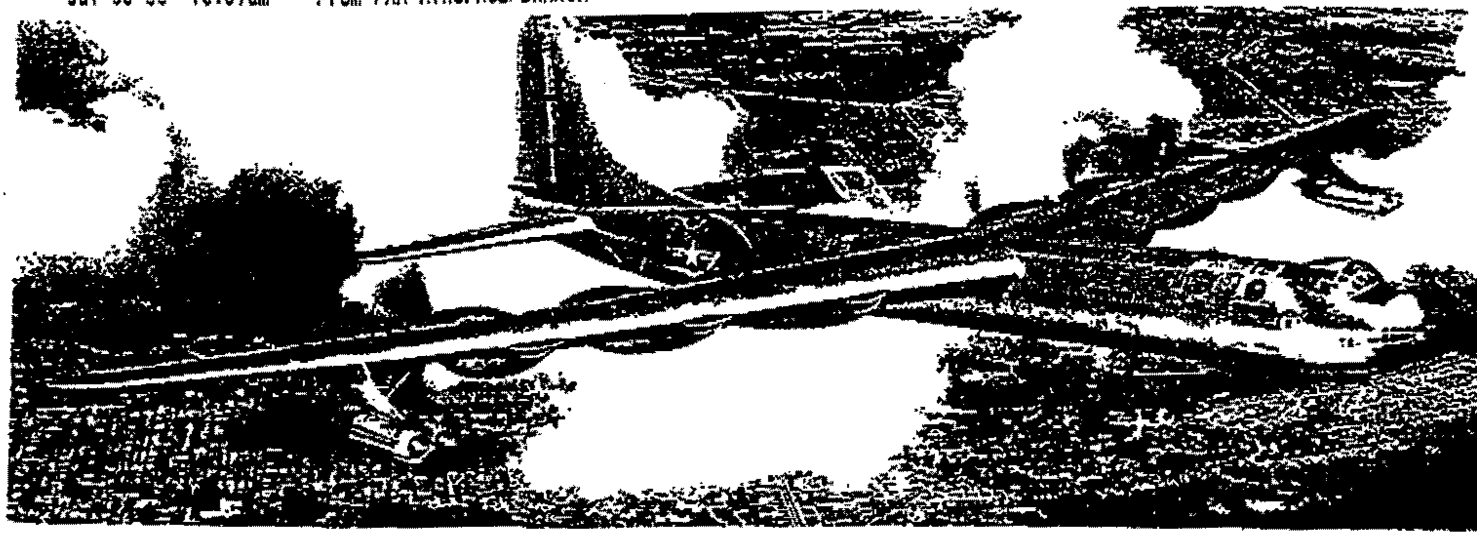
Vi [redacted] y Rd

2000 VIC SALEM PROJECT MIDWAY
ROAD



B-36 THE PEACEMAKER

BRUCE BEARD - SENIOR OBSTRUCTION EVALUATION SPECIALIST
FEDERAL AVIATION ADMINISTRATION
SOUTHWEST REGIONAL OFFICE
AIR TRAFFIC AIRSPACE BRANCH
FORT WORTH TX. 76193-0520

OFFICE NUMBER: 817 / 222-5536
FAX NUMBER: 817 / 222-5981

TO: ROGER D

FAX NUMBER: 972-418-6388

REFERENCE: ADS AIRPORT

MESSAGE:

NUMBER OF PAGES (INCLUDING COVER PAGE): 8

A HARD COPY ~~WILL~~ WILL NOT BE SENT.

Federal Aviation Administration
Southwest Region
Air Traffic Division, ASW-520
Fort Worth, TX 76193-0520

AERONAUTICAL STUDY
No: 99-ASW-2295-OE

ISSUED DATE: 07/07/99

ROGER DAHLIN
ROGER DAHLIN ARCHITECT
2207 VIA DEL NORTE CIRCLE
CARROLLTON TX 75006

** FEASIBILITY REPORT **

The Federal Aviation Administration has conducted a limited aeronautical review concerning the feasibility of a structure described as follows:

Description:	FEASIBILITY STUDY / BUILDING POINT A / ADJACENT TO ADDISON AIRPORT
Location:	CARROLLTON TX
Latitude:	32-58-27.44 NAD 83
Longitude:	096-50-28.00
Heights:	35 feet above ground level (AGL) 660 feet above mean sea level (AMSL)

The results of this review can be found on the attached page(s).

NOTE: THE RESULTS OF OUR LIMITED REVIEW IS NOT AN OFFICIAL DETERMINATION OF FINDINGS BUT ONLY A REPORT BASED ON THE GENERAL OR ESTIMATED INFORMATION SUPPLIED FOR THE STRUCTURE. ANY FUTURE, OFFICIAL AERONAUTICAL STUDY MAY REVEAL DIFFERENT RESULTS.

If we can be of further assistance, please contact our office at 817-222-5534. On any future correspondence concerning this matter, please refer to Aeronautical Study Number 99-ASW-2295-OE.

Bruce C. Beard

Bruce C. Beard
Specialist, Airspace Branch

(FSB)

Attachment

ATTACHMENT SHEET
AERONAUTICAL STUDY NUMBER 99-ASW-2295-OE
CARROLLTON, TEXAS

POINT A

THIS IS NOT A FORMAL DETERMINATION

PART 77 = TITLE 14 OF THE CODE OF FEDERAL REGULATIONS, PART 77
AGL = ABOVE GROUND LEVEL / AMSL = ABOVE MEAN SEA LEVEL
SIAP = STANDARD INSTRUMENT APPROACH PROCEDURE
NM = NAUTICAL MILE

This informal feasibility report used the data either submitted by you, determined by this office or a combination of both and is shown on Page 1.

- The proposed site would be located approximately 622 feet west and perpendicular to Runway 15 at the Addison Airport.
- Based on the requirements contained in part 77, notice to the FAA would be required.
- This notice should be submitted to our office at least 60 days prior to the start of any construction.
- In addition, it does appear that the proposal would penetrate the following obstruction standards contained in part 77:

• Section 77.23 (a)(5) by 6 feet - a height exceeding the transition surface as applied to Runway 15 at the Addison Airport. A structure height of 29 feet AGL / 654 feet AMSL would not exceed this obstruction standard.

- Preliminary review indicates that a structure at your proposed location and a height no greater than 35 feet AGL / 660 feet AMSL is feasible. This is based on a site elevation of 625 feet.
- For a structure height of 42 feet AGL, the site elevation would have to be 618 feet AMSL (618 + 42 = 660)
- This is ***NOT*** a formal determination but only a report based on the information furnished this office. Please keep in mind that there is always a possibility that the final outcome of a formal aeronautical study might prove to be different from the results of this informal feasibility study.

- PLEASE NOTE THAT YOUR PROPOSAL AT A HEIGHT OF 35 FEET AGL / 660 FEET AMSL WILL REQUIRE A FORMAL AERONAUTICAL STUDY. A FORMAL AERONAUTICAL STUDY TAKES APPROXIMATELY 90 TO 120 DAYS TO COMPLETE, SO YOU WILL NEED TO PLAN ACCORDINGLY.

- This report does not relieve the sponsor of any compliance responsibilities relating to laws, ordinances, or regulations of any Federal, state, or local governmental bodies.

- This informal feasibility report does not supersede or override any state, county, or local laws or ordinances.

- If you do not agree with the coordinates, elevation, heights, or the results of this report, please contact me at 817-222-5536.

- Based on the unofficial nature of this study, the FAA shall not be held responsible for any type of commitment entered into by the sponsor base solely on this informal feasibility report.

- Please refer to **Aeronautical Study Number 99-ASW-2295-OE** on any future correspondence concerning this feasibility report or if you do file formal notice with the FAA concerning the structure.

Additional Comments:

The existing building located south of your proposal has an overall height of 660 feet AMSL. A site elevation of 618 feet AMSL + a building height of 42 feet AGL = an overall height of 660 feet AMSL. The overall height is the magic number and not the AGL height of the structure. The overall height is dependant on the site elevation and structure height.

Based on the Addison 7.5" Quadrangle Chart, the maximum site elevation at your location 625 feet AMSL. A site elevation of 625 feet AMSL + a building height of 35 feet = an overall height of 660 feet AMSL.

For a site located this close to the airport, we request that when submitting your notice to this office, the exact location and site elevation of Point A be determined by a survey. The location will need to be in latitude/longitude.

It is certainly possible that with surveyed data, a height of 42 feet AGL might be acceptable. However, this cannot be determined without conducting a formal study.

Federal Aviation Administration
Southwest Region
Air Traffic Division, ASW-520
Fort Worth, TX 76193-0520

AERONAUTICAL STUDY
No: 99-ASW-2296-OE

ISSUED DATE: 07/07/99

ROGER DAHLIN
ROGER DAHLIN ARCHITECT
2207 VIA DEL NORTE CIRCLE
CARROLLTON TX 75006

** FEASIBILITY REPORT **

The Federal Aviation Administration has conducted a limited aeronautical review concerning the feasibility of a structure described as follows:

Description:	FEASIBILITY STUDY / BUILDING POINT B / ADJACENT TO ADDISON AIRPORT
Location:	CARROLLTON TX
Latitude:	32-58-31.85 NAD 83
Longitude:	096-50-30.33
Heights:	35 feet above ground level (AGL) 660 feet above mean sea level (AMSL)

The results of this review can be found on the attached page(s).

NOTE: THE RESULTS OF OUR LIMITED REVIEW IS NOT AN OFFICIAL DETERMINATION OF FINDINGS BUT ONLY A REPORT BASED ON THE GENERAL OR ESTIMATED INFORMATION SUPPLIED FOR THE STRUCTURE. ANY FUTURE, OFFICIAL AERONAUTICAL STUDY MAY REVEAL DIFFERENT RESULTS.

If we can be of further assistance, please contact our office at 817-222-5534. On any future correspondence concerning this matter, please refer to Aeronautical Study Number 99-ASW-2296-OE.

Bruce C Beard

Bruce C. Beard
Specialist, Airspace Branch

(FSB)

Attachment

ATTACHMENT SHEET
AERONAUTICAL STUDY NUMBER 99-ASW-2296-OE
CARROLLTON, TEXAS

POINT B

THIS IS NOT A FORMAL DETERMINATION

PART 77 = TITLE 14 OF THE CODE OF FEDERAL REGULATIONS, PART 77
AGL = ABOVE GROUND LEVEL / AMSL = ABOVE MEAN SEA LEVEL
SIAP = STANDARD INSTRUMENT APPROACH PROCEDURE
NM = NAUTICAL MILE

This informal feasibility report used the data either submitted by you, determined by this office or a combination of both and is shown on Page 1.

- The proposed site would be located approximately 622 feet west and perpendicular to Runway 15 at the Addison Airport.
- Based on the requirements contained in part 77, notice to the FAA would be required.
- This notice should be submitted to our office at least 60 days prior to the start of any construction.
- In addition, it does appear that the proposal would penetrate the following obstruction standards contained in part 77:

• Section 77.23 (a)(5) by 6 feet - a height exceeding the transition surface as applied to Runway 15 at the Addison Airport. A structure height of 29 feet AGL / 654 feet AMSL would not exceed this obstruction standard.

- Preliminary review indicates that a structure at your proposed location and a height no greater than 35 feet AGL / 660 feet AMSL is feasible. This is based on a site elevation of 625 feet.
- For a structure height of 42 feet AGL, the site elevation would have to be 618 feet AMSL (618 + 42 = 660)
- This is ***NOT*** a formal determination but only a report based on the information furnished this office. Please keep in mind that there is always a possibility that the final outcome of a formal aeronautical study might prove to be different from the results of this informal feasibility study.

- PLEASE NOTE THAT YOUR PROPOSAL AT A HEIGHT OF 35 FEET AGL / 660 FEET AMSL WILL REQUIRE A FORMAL AERONAUTICAL STUDY. A FORMAL AERONAUTICAL STUDY TAKES APPROXIMATELY 90 TO 120 DAYS TO COMPLETE, SO YOU WILL NEED TO PLAN ACCORDINGLY.

- This report does not relieve the sponsor of any compliance responsibilities relating to laws, ordinances, or regulations of any Federal, state, or local governmental bodies.

- This informal feasibility report does not supersede or override any state, county, or local laws or ordinances.

- If you do not agree with the coordinates, elevation, heights, or the results of this report, please contact me at 817-222-5536.

- Based on the unofficial nature of this study, the FAA shall not be held responsible for any type of commitment entered into by the sponsor base solely on this informal feasibility report.

- Please refer to **Aeronautical Study Number 99-ASW-2296-OE** on any future correspondence concerning this feasibility report or if you do file formal notice with the FAA concerning the structure.

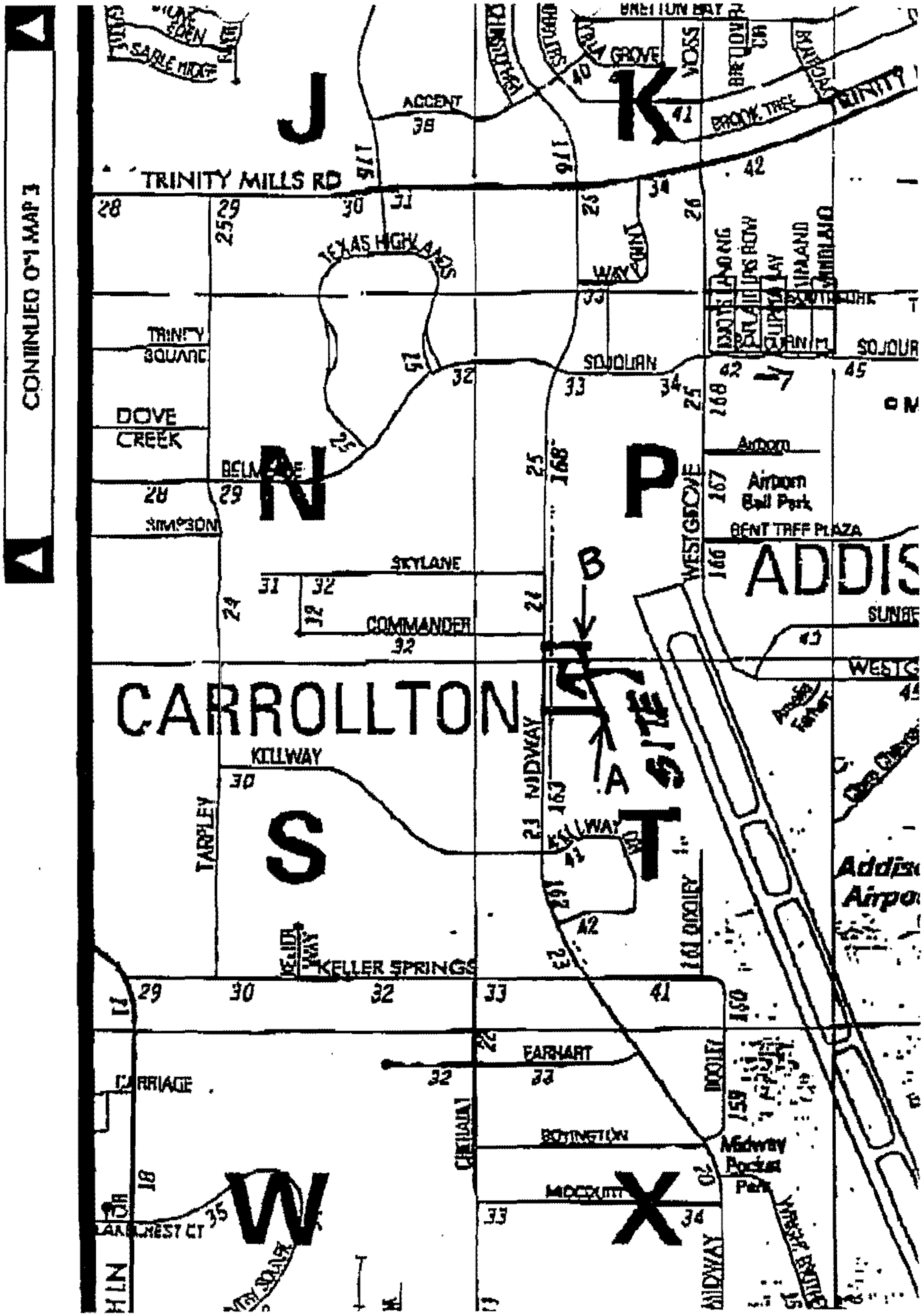
Additional Comments:

The existing building located south of your proposal has an overall height of 660 feet AMSL. A site elevation of 618 feet AMSL + a building height of 42 feet AGL = an overall height of 660 feet AMSL. The overall height is the magic number and not the AGL height of the structure. The overall height is dependent on the site elevation and structure height.

Based on the Addison 7.5" Quadrangle Chart, the maximum site elevation at your location 625 feet AMSL. A site elevation of 625 feet AMSL + a building height of 35 feet = an overall height of 660 feet AMSL.

For a site located this close to the airport, we request that when submitting your notice to this office, the exact location and site elevation of Point B be determined by a survey. The location will need to be in latitude/longitude.

It is certainly possible that with surveyed data, a height of 42 feet AGL might be acceptable. However, this cannot be determined without conducting a formal study.


CONTINUED ON MAP 3
TRINITY MILLS RD
ACCENT
BROOK TREE
BRETTON BAY
VOSS
TEXAS HIGHLANDS
TRINITY SQUARE
DOVE CREEK
BELMEADE
SIMPSON
SOJOURN
Airborn
Airborn Ball Park
BENT TREE PLAZA
WESTGROVE
ADDIS
SKYLANE
COMMANDER
CARROLLTON
B
A
SITE
KILLWAY
MIDWAY
TARPLEY
KELLER SPRINGS
DOOLEY
EARHART
CARRIAGE
BOYINGTON
Midway Pocket Park
LAKECREST CT
Addis
Airpo
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N
P
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X

GEOTECHNICAL EVALUATION
PROPOSED OFFICE/WAREHOUSE BUILDING
MIDWAY ROAD
ADDISON, TEXAS

Vic Sahm Project

MAXIM
TECHNOLOGIES INC

Prepared for:
S & B Investments
Dallas, Texas

GEOTECHNICAL EVALUATION
PROPOSED OFFICE/WAREHOUSE BUILDING
MIDWAY ROAD
ADDISON, TEXAS

Vic Sahm Project

Prepared by:
MAXIM TECHNOLOGIES, INC.
2200 Gravel Drive
Fort Worth, Texas 76118
(817) 589-7211

Report No. 9912696

February 8, 2000

MAXIM
TECHNOLOGIES INC

February 8, 2000

Mr. Vic Sahm
S&B Investments
P.O. Box 700008
Dallas, Texas 75370

Re: Geotechnical Evaluation
Proposed Office/Warehouse Building
Midway Road
Addison, Texas
Maxim Project No. 9912696

Dear Mr. Sahm:

Please find enclosed our report summarizing the results of the geotechnical evaluation performed at the above referenced project. We trust the recommendations derived from this investigation will provide you with the information necessary to achieve a quality project in a timely and cost efficient manner.

As your project progresses through the design and construction phases, we recommend that Maxim Technologies, Inc. be retained to provide geotechnical/construction materials engineering, testing, and inspection services for this project.

We thank you for the opportunity to provide you with our professional services. If we can be of further assistance, please do not hesitate to contact us.

Sincerely,

MAXIM TECHNOLOGIES, INC.

T. Neill Lawrence, E.I.T.
Geotechnical Division

Doyle L. Smith, Jr., P.E.
Vice President

STATE OF TEXAS
DOYLE L. SMITH, JR.
56542
REGISTERED
PROFESSIONAL ENGINEER
2/10/00

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Proposed Office/Warehouse Building
Midway Road
Addison, Texas

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Addison, Texas

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APPENDIX

GEOTECHNICAL EVALUATION
PROPOSED OFFICE/WAREHOUSE BUILDING
MIDWAY ROAD
ADDISON, TEXAS

1.0 PROJECT INFORMATION

The project will involve the construction of a new office/warehouse building located on the vacant tract directly north of the existing office/warehouse building located at 16400 Midway Road in Addison, Texas. The proposed office/warehouse building is expected to have a building footprint of approximately 70,000 square feet. Detailed structural information was not available, however, the maximum column loads for the building are expected to be in the 100 kip range. It is assumed that fill required to develop the planned finished floor site grades will be 2 to 4 feet. One (1) to Seven (7) feet of fill materials consisting of clay intermixed with limestone and concrete was encountered at this site. Further limited investigation of the fill materials will be made and submitted in an addendum report within the next few days. If the details of the proposed construction are different than stated herein, please contact our office to evaluate the potential impact to the recommendations presented in this report.

2.0 SCOPE OF INVESTIGATION

Our services for this project were performed in general conformance with our proposal dated November 10, 1999 (Proposal No. 9-11-05. The purposes of this geotechnical evaluation were to: 1) explore the subsurface conditions at the site, 2) evaluate the pertinent engineering properties of the subsurface materials, 3) provide recommendations concerning suitable types of foundation systems for the proposed structures, 4) provide pavement system recommendations, and 5) provide comments and recommendations concerning construction guidelines for earthwork operations including excavation and fill placement.

3.0 FIELD OPERATIONS

Eleven (11) test borings were drilled at the approximate locations shown on the Boring Location Diagram on Figure 1. Seven (7) borings were advanced to depths ranging from 20 to 25 feet each in the vicinity of the proposed building while four (4) borings were advanced to a depth of 5 feet each in the vicinity of the proposed parking area and drives. The results of the boring program are presented on the Logs of Borings, Figures 2 through 12.

A truck-mounted continuous flight auger drill rig was used to advance the borings and to obtain samples for laboratory evaluation. Undisturbed samples of the cohesive soil were obtained at intermittent intervals with standard, thin-walled, seamless tube samplers. These samples were extruded in the field, logged, sealed and packaged to protect them from disturbance and to maintain their in-situ moisture content during transportation to our laboratory.

The bearing properties of the limestone formation encountered was evaluated by the Texas Department of Transportation's (TxDOT) Cone Penetrometer Test. This test consists of measuring the penetration of a 3-inch diameter cone driven with a 170-pound hammer falling 24 inches. The results of these tests are tabulated on the boring logs.

4.0 LABORATORY TESTING

The project geotechnical engineer examined the samples recovered during the field exploration program at our laboratory. Select samples were then subjected to laboratory tests under the supervision of this engineer. The in-situ unit weight, moisture content, and liquid and plastic limits of the select soil samples were measured to evaluate the potential volumetric change of the different strata and as an indication of the uniformity of the material. Unconfined compression tests were performed to estimate the unconfined compressive strength of the soil. Hand penetrometer tests were performed to provide an indication of the variation of soil strength with depth. The test results are tabulated on the Logs of Boring.

5.0 SURFACE AND SUBSURFACE SITE CONDITIONS

5.1 Site Geology

As shown on the Dallas sheet of the Geologic Atlas of Texas, the site is located in an area underlain by deposits of the Austin Chalk Formation. The Austin Chalk Formation consists of moderately to highly plastic overburden clays underlain by limestone.

5.2 Subsurface Conditions

Subsurface conditions encountered in the borings, including descriptions of the various strata and their depths and thickness', are presented on the Logs of Boring. Note that depth on all borings refers to the depth from the existing grade or ground surface present at the time of the investigation. Boundaries between the various soil types are approximate.

The subsurface soils encountered at this site consisted of both fill materials and native clay soils. The fill material consists of clay soils intermixed with limestone fragments and concrete within the upper one (1) to seven (7) feet in the vicinity of borings B-2 through B-7. Based on field and laboratory tests, the fill material appear to have been satisfactorily compacted. However, we recommend that all areas containing fill be proofrolled as described in Section 7.2 of this report.

The native soils encountered at this site consisted of 7 to 14 feet of moderately to highly plastic dark brown, brown, yellowish brown, and gray clays underlain by tan weathered limestone of the Austin Chalk formation. The primary gray limestone stratum was encountered at depths ranging from 11 to 22 feet below existing grade and extended to the termination of the deepest borings 25 feet below existing grade.

5.3 Groundwater

The borings were advanced with continuous flight auger drilling equipment. This method allows relatively accurate short term groundwater observations to be made while drilling. Subsurface perched groundwater seepage was encountered at depths ranging from 5 to 14 feet below existing grade at the time of this investigation at boring locations B-1, B-3, B-4, and B-5. An accurate determination of the uppermost water bearing zone would require the installation of groundwater monitoring wells and a water level monitoring program extending over several months. Furthermore, the presence and magnitude of perched groundwater will fluctuate seasonally due to variations in the amount of precipitation, evapotranspiration, upper elevation of the aquitard (limestone formation at this site) and the surface water runoff characteristics of this site and the surrounding area. The presence of perched groundwater should be verified prior to construction that would be adversely impacted by subsurface perched groundwater, such as temporary casing of drilled piers during installation.

6.0 ANALYSIS AND RECOMMENDATIONS

6.1 Soil Movements

The Texas Department of Transportation's (TxDOT) empirical method of predicting Potential Vertical Rise (PVR) was utilized in the development of the foundation design criteria associated with soil movement. PVR values obtained using this method assume that the supporting soil is not subject to free water sources and, as a result, never becomes fully saturated and never reaches its full swell potential. PVR values using this method are typically much lower than values obtained from laboratory swell tests. When this design method is used it is imperative that all potential free water sources are eliminated in order to prevent excessive upward movement caused by soil swelling. It is also imperative that measures be taken during design and construction to reduce the risk of free water sources near the foundation (see Appendix A of this report) and that the owner be advised of the importance of maintaining the conditions described in Appendix A of this report.

The clay deposits are highly expansive and have a high shrink/swell potential within the normal zone of seasonal moisture change. Potential Vertical Rise (PVR) calculations were performed using TxDOT Method 124-E, assuming a "dry" soil moisture condition to estimate the swell potential of the soil. The PVR value was estimated to be approximately 4.0 inches.

Considerably more upward movement than the estimated above potential soil movement will occur in areas where water is allowed to pond near or beneath the structure for extended periods due to poor drainage, leaking utility lines, percolation in recessed landscaped areas, or leaking sprinkler lines. The soil conditions may also differ from those encountered at the boring locations, which will influence the estimated soil movement.

6.2 Foundation System Recommendations

Due to the highly expansive nature of the subsurface clays encountered at this site, structural loads of the proposed building should be supported by straight drilled shafts bearing into the gray Austin Chalk stratum encountered at depths ranging from 11 to 22 feet below existing grade.

We recommend an allowable end bearing pressure of 50,000 psf be used for design for shafts bearing a minimum of two (2) feet into the hard gray limestone stratum. A skin friction value of 5,000 psf is recommended for compressive loads and 3,000 psf for uplift load resistance.

Drilled shafts should have a minimum penetration depth of 2 feet into the hard gray limestone to develop the recommended end bearing values. Skin friction and uplift resistance may be considered after the minimum penetration of 2 feet into the hard gray limestone.

Since some variation in the depth of drilled piers may be required due to the variable depth and potential weathered condition of the upper portion of the gray Austin Chalk formation, bid and contract documents should include pay items for constructing drilled shafts on a unit price basis. Due to the presence of perched groundwater encountered during our investigation, bid and contract documents should also include pay items for the use of temporary casing on a unit price basis for installation of drilled piers where casing is necessary. When estimating total pier depths and developing unit costs for drilled pier installation for bidding purposes, the following items should be adequately addressed:

- The drilling resistance of the Austin Chalk limestone
- The variable depth of the gray limestone formation, as noted on the individual boring logs
- The surface elevation of each boring location relative to the finished floor elevation for the proposed building
- Potential perched groundwater seepage encountered during drilled pier installation.

6.2.1 Drilled Shaft Supported Grade Beams

All grade beams or wall panels should be supported by drilled shafts and a minimum void space of eight (8) inches provided between the bottom of these members and the subgrade. This void will serve to reduce distress resulting from swell pressures generated by the near surface expansive clays.

Structural cardboard boxes are one acceptable means of providing this void beneath cast-in-place beams. A soil retainer should be provided to help prevent in-filling of the void. Care must be exercised during concrete placement to avoid collapsing the cardboard void boxes. The grade beam or wall panel excavations around the perimeter of the building should be carefully backfilled with on-site soils.

6.2.2 Group Effects On Straight Drilled Shafts

In order to develop full load carrying capacity in end bearing, adjacent shafts should have a minimum clear spacing of two (2) times the diameter of the larger shaft. Closer spacing may require a reduction in skin friction. Shafts spaced closer than three shaft diameters should be evaluated on a case by case basis by the geotechnical engineer.

6.2.3 Straight Drilled Shaft Soil Uplift Loads

Straight drilled shafts should penetrate the hard gray limestone a sufficient amount to provide resistance to potential uplift forces caused by soil swelling. Uplift loads will be induced on the shafts by soil heave in the overlying clays. The magnitude of these loads varies with the shaft diameter, free water sources, soil parameters, the depth of the clays acting on the shaft, and particularly the in-situ moisture levels at the time of construction. These pressures can be approximated at this site by assuming a uniform uplift pressure of 1,500 psf acting on the shaft perimeter for a shaft length of 10 feet. The shafts should have sufficient continuous vertical reinforcement extending to the base of the shafts to resist the computed uplift loads. The sustained structure dead load may also be considered to resist soil uplift pressures.

6.3 Drilled Shaft Construction Considerations

Excavations for the shafts should be maintained in the dry. Based on our field investigation groundwater seepage will likely be encountered during installation of some of the drilled shafts, especially if construction proceeds during wet periods of the year. In some cases, rapid placement of steel and concrete may permit shaft installation to proceed without the need for casing, however, provisions for temporary casing should be included in the contract documents. Seepage rates that result in excessive standing water in the bottom of the shafts at the time of concrete placement will require pumping and/or the use of temporary casing for installation of these shafts. If required, temporary casing should be seated in the hard dark gray shale and properly sealed below the seepage zone to prevent excessive seepage into the drilled shaft excavation. Care must then be taken that a sufficient head of plastic concrete is maintained within the casing during extraction. If casing is required, specified pier penetrations should be measured from the bottom of the casing.

Concrete used for the shafts should have a slump of 5 inches plus or minus 1 inch and be placed in a manner to avoid striking the reinforcing steel and walls of the shaft during placement. Complete installation of individual shafts should be accomplished within an 8 hour period in order to prevent deterioration of bearing surfaces. The drilling of individual shafts should be excavated in a continuous operation and concrete placed as soon as practical after completion of the drilling. No shaft should be left open for more than 8 hours.

We recommend that Maxim Technologies, Inc., be retained to observe and document the drilled pier construction. The geotechnical engineer, or his representative, should document the shaft diameter, depth, cleanliness, plumbness of the shaft, the type of bearing material and casing installations. Significant deviations from the specified or anticipated conditions should be reported to the owner's representative, the structural engineer, and the geotechnical engineer. The drilled pier excavation should be observed after the bottom of the hole is cleaned of any mud or extraneous material, and dewatered, if necessary.

6.4 Floor Slab Systems

Due to the potential for excessive upward slab movements, designed system performance and constructability (schedule impact and construction cost) the floor slabs should consist of a slab-on-grade that is placed on 1) select fill materials or 2) select fill soils placed over a stabilized subgrade that has been pre-swelled by water pressure injection.

6.4.1 Slab-On-Grade Floor System (Select Fill Only)

The presence of expansive clay soils at this site will result in differential movement of slab-on-grade floor slabs, therefore, site preparation work will be required in order to lower the potential soil movement.

We recommend that the following building pad preparation work be performed for the building in order to reduce potential differential floor movements to 1.00 to 1.25 inches.

1. Adjust the building pad subgrade as required to allow the placement of at least 5 feet of "select" fill beneath the floor slab (not including the thickness of the moisture retarding layer and concrete floor slab). The grade adjustment, and "select" fill placement should extend a minimum of 5 feet beyond the building perimeter and beneath adjacent sidewalks and entry slabs. Proofroll with exposed subgrade. Proofrolling can generally be accomplished using a heavy (25 ton or greater total weight) pneumatic tired roller making several passes over the area. Where soft or compressible zones are encountered, these areas should be removed to stiff subgrade. Any resulting void areas should be backfilled to finished subgrade in 6 inch compacted lifts compacted to 95 percent of maximum dry density as determined by ASTM D 698 between 0 and +5 percentage points of its optimum moisture content.

2. Scarify, rework, and recompact the upper 8 inches of the exposed subgrade. The scarified soils should be recompacted to 95 percent of the maximum density as determined by ASTM D 698 between 0 and +5 percentage points of its optimum moisture content.

3. The upper 5 feet of pad fill should consist of non-expansive select fill having a PI of 5 to 15. Compact at –2 to +3% above optimum to a minimum of 95% Standard Proctor Density. The upper 2 feet of backfill in unpaved areas near the building should consist of on-site cay compacted to 95 percent (to minimum water infiltration into the select fill).

4. The subgrade moisture content within the building pad must be maintained until all slabs have been constructed.

6.4.2 Slab-On-Grade Floor System (Select Fill and Water Pressure Injection)

Site preparation work will be required in order to lower the potential soil movements to a tolerable level. Water pressure injection stabilization to pre-swell the clay soils can be performed at this site to reduce the potential soil movement. We recommend that a guaranteed maximum price be obtained from an injection subcontractor to reduce the average swell to less than one (1) inch for a 10 foot depth of treatment.

Water injection stabilization is a time consuming process and should be considered during schedule planning since several injection passes will be required. Recommendations for water pressure injection procedures are presented below.

1. Adjust the building pad subgrade as required to allow the placement of at least 2.5 feet of "select" fill beneath the floor slab (not including the thickness of the moisture retarding layer and concrete floor slab. The grade adjustment, "select" fill placement and water injection should extend a minimum of 5 feet beyond the building perimeter and beneath adjacent sidewalks and entry slabs.

2. Water inject to a ten (10) foot depth. The water injection process should be continued until the desired PVR has been achieved. Injection stabilization should be performed 5 feet beyond building lines, entries and adjacent sidewalks. For a 1.0 inch PVR, the acceptance criteria should be based on the results of volumetric swell tests, moisture content tests, and hand penetrometer readings perform for each test boring (the loading for the swell tests should include the select fill surcharge loads). Multiple injections will be required. Specifications for this work are included in Appendix B.

3. After completion and acceptance of injection stabilized pad, remove ponding water, aerate, proofroll, rework as needed and compact at +2% to +5% above optimum moisture content to a minimum density of 93% Standard Proctor Compaction.

4. The upper 2.5 feet around the building perimeter and upper two feet in the building interior of pad fill should consist of non-expansive select fill having PI of 4 to 15. Compact at -2 to +3% above optimum to a minimum of 95% Standard Proctor density. The upper 8 inches of fill in unpaved areas adjacent to building should consist of on-site compacted clay to minimize water infiltration into the select fill.

5. Moisture condition of all earthwork and completed pad must be maintained until all slabs are in place.

A set of General Specifications for this process is presented in Appendix B of this report. Compliance with these specifications is essential if maximum benefits are to be gained. We recommend the injection process be observed on a full time basis by qualified Maxim personnel.

A polyethylene moisture barrier is recommended below the building floor slabs where floor coverings or painted floor surfaces will be applied with products which are sensitive to moisture or if products stored on the building floors are sensitive to moisture. Procedures for installation of vapor barriers are recommended in ACI 302 Section 2.4.1.

7.0 EARTHWORK GUIDELINES

7.1 Site Grading and Drainage

All grading should provide positive drainage away from the structure and should prevent water from collecting or discharging near the foundations. Water must not be permitted to pond adjacent to the structure during or after construction.

Surface drainage gradients should be designed to divert surface water away from the building and edges of pavements and towards suitable collection and discharge facilities. Unpaved areas and permeable surfaces should be provided with steeper gradients than paved areas. Surface drainage gradients within 10 feet of the building should be constructed with a minimum slope of 1 percent for paved areas and 3 percent for unpaved areas.

The roof should be provided with gutters and downspouts to prevent the discharge of rainwater directly onto the ground adjacent to the building foundations. Downspouts should discharge directly into storm drains or drainage swales, if possible. Roof downspout and surface drain outlets should discharge into erosion-resistant areas, such as paving or rock riprap.

Water permitted to pond in planters, open areas, or areas with unsealed joints next to the structure can result in on-grade slab or pavement movements that exceed those indicated in this report. It is emphasized that predictions of moisture related differential movements indicated in this report are based on empirical calculations and previous experience. In some cases movements can exceed those predicted, particularly when unusual sources of water become available to the underlying clay.

Exterior sidewalks and pavements will be subject to some post construction movement. Flat grades should be avoided. Where concrete pavement is used, joints should be sealed to prevent the infiltration of water. Since some post construction movement of pavement and flatwork may occur, joints particularly around the building should be periodically inspected and resealed where necessary.

7.2 Site Preparation for Controlled Placement of Fill

Subgrade preparation should include the removal of the existing structures, including all foundations, floor slabs, pavements, and any other below grade structures. Any underground utilities that are not going to be reused should be removed and capped at the property lines, or rerouted around the site and reconnected. All topsoil, surface vegetation, tree root balls, and any other deleterious materials should also be removed from the planned pavement areas.

Following removal operations, the subgrade should be proofrolled under the observation of the geotechnical engineer or a qualified engineering technician. Proofrolling should be accomplished using a heavy (25 ton or greater total weight) pneumatic tired roller making several passes over the area. Any soft or unstable soil that is encountered should be removed to a firm subgrade. The over-excavation should then be backfilled to finished subgrade with suitable fill. The backfill should be placed in 6 inch lifts and compacted to at least 95 percent of the Standard Proctor density at a moisture content between 0 and +5 percent of the optimum moisture value.

7.3 Select Fill

Select fill should consist of sandy clay to clayey sand with a liquid limit of 32 or less and a plasticity index between 4 and 15. The fill should be spread in loose lifts, less than 9 inches thick, and uniformly compacted to at least 95 percent of Standard Proctor density at a moisture content within 3 percent of the optimum moisture value. The moisture content of the completed pad must be maintained during construction until all slabs have been constructed (including pavement slabs).

7.4 On-Site Clay Fill

The on-site surficial clay may be used as fill in pavement and landscaped areas. The fill should be free of surficial vegetation or debris. Clay fill should be spread in loose lifts, less than 9 inches thick, and uniformly compacted to at least 95 percent of the Standard Proctor density at a moisture content between optimum and 5 percent above optimum.

8.0 PAVEMENT RECOMMENDATIONS

We assume that a typical automobile is representative of essentially all of the traffic on the proposed parking area pavements. We further assume that only occasional heavy to medium truck traffic from primarily waste disposal trucks and delivery trucks will be present on the parking area pavements. The following recommendations are based upon these assumed conditions.

8.1 Pavement Design Considerations

The clays that are present on this site are active and will lose strength with the increases in moisture content that normally occur beneath pavements. However, the support characteristics of these clay deposits can be improved through lime stabilization. Lime stabilization consists of mixing the subgrade soil with hydrated lime in order to improve the support characteristics of the soil and to provide a firm, uniform subgrade beneath the pavement.

We typically recommend lime stabilization of subgrade used for support of all asphalt pavements (with the exception of soluble sulfate bearing clays), that will be subject to moderate to high traffic loads. Recommendations for subgrade stripping and proofrolling, lime stabilization, and subgrade recompaction are presented in the following sections.

If lime stabilization is performed the application rate should be determined by performing a lime series test on the exposed subgrade after the pavement area has achieved approximate final grade. Lime stabilization criteria should be established using current TxDOT (Item 260) and/or applicable NCTCOG (Item 4.6) specifications.

8.2 Recommended Pavement Sections Considerations

Recommendations for both Portland Cement concrete pavement and asphalt pavement are provided in the following Tables 1 and 2. The pavement sections provided in Table 1 are recommended for areas that will be subject to relatively light traffic loads, such as the parking stalls for automobiles.

The pavement sections provided in Table 2 are recommended for areas that will be subject to moderate traffic loads, such as drives and areas subject to occasional truck traffic.

TABLE 1
LIGHT DUTY PAVEMENT SECTION

LIGHT DUTY PAVEMENT (Parking Stalls)
HMAC SECTIONS 2.0 inches Asphaltic Concrete Surface Course Type D 3.0 inches Asphaltic Concrete Base Course Type B 6.0 inches Lime Treated Subgrade **PCC SECTION** 5.0 inches Portland Cement Concrete 6.0 inches Scarified and Compacted Subgrade

TABLE 2
HEAVY DUTY PAVEMENT SECTION

HEAVY DUTY PAVEMENT (Drive Way and Drive Approaches with Occasional Truck Traffic)
HMAC SECTIONS 3.0 inches Asphaltic Concrete Surface Course Type D 4.5 inches Asphaltic Concrete Base Course Type B 6.0 inches Lime Treated Subgrade **PCC SECTION** 7.0 inches Portland Cement Concrete 6.0 inches Scarified and Compacted Subgrade

Asphaltic concrete pavement should be placed in accordance with Item 340 of TxDOT's Standard Specification, 1995 edition. The surface course asphaltic concrete should comply with Type D of Item 340 and the base course asphaltic concrete should comply with Type B of TxDOT Item 340.

The Portland Cement concrete should have a minimum 28 day compressive strength of 3,500 psi. Concrete quality will be important in order to produce the desired flexural strength and long term durability. Assuming a nominal maximum aggregate size of 1 to 1-1/2 inches, we recommend that the concrete have entrained air of 5 percent ($\pm$ 1%) with a maximum water cement ratio of 0.50.

Proper joint placement and design are critical to pavement performance. Load transfer at all longitudinal joints and maintenance of watertight joints should be accomplished by use of tie bars. Control joints should be sawed as soon as possible after placing concrete before shrinkage cracks occurs. Joints should also be properly cleaned and sealed as soon as possible to avoid infiltration of water, small gravel, etc.

Our previous experience indicates that joint spacing on 12 to 15 foot centers have generally performed satisfactorily. A 12 foot spacing is preferred. It is recommended that the concrete pavement be reinforced with No. 3 bars on approximately 18-inch centers in each direction or equivalent reinforcing steel. We recommend that the perimeter of the pavement area have a stiffening curb section to prevent possible distress due to heavy wheel loads near the edge of the pavement and to provide channelized drainage.

8.3 Other Pavement Considerations

All joints and pavements should be inspected at regular intervals to ensure proper performance and to reduce the potential for crack propagation. The site soil is active and differential heave of the pavement could occur. The service life of the pavement may be reduced due to water infiltration into the subgrade soil through heave induced cracks in the pavement section. This will result in softening and loss of strength of the subgrade soils. A regular maintenance program to seal paving cracks will help prolong the service life of the pavement.

The life of the pavement can be increased with proper drainage. Areas should be graded to prevent ponding adjacent to curbs or pavement edges. Curb areas should be backfilled as soon as possible after the concrete has set, preferably with the on-site clay soil. Backfill materials that could hold water behind the curb should not be permitted. Flat pavement grades should be avoided.

9.0 CONSTRUCTION MONITORING AND TESTING

Many problems can be avoided or solved in the field if proper monitoring and testing services are provided. It is recommended that all foundation excavation, proofrolling, site and subgrade preparation, and subgrade stabilization be monitored by a qualified engineering technician. Field density and moisture content determinations should be made on each lift of fill with a minimum of 1 test per lift per 5,000 square feet in the building pad, 1 test per lift per 100 linear feet of utility trench backfill, and 1 test per lift per 10,000 square feet in other fill areas. Inspection should be performed prior to and during concrete placement operations. We employ a group of experienced, well-trained technicians for inspection and construction materials testing who would be pleased to assist you on this project.

10.0 LIMITATIONS

The professional services that have been performed, the findings obtained, and the recommendations prepared were accomplished in accordance with currently accepted geotechnical engineering principles and practices. If there are any unusual conditions differing significantly from those described herein, Maxim Technologies, Inc. should be notified to review the effects on the performance of the recommended foundation system.

The recommendations given in this report were prepared exclusively for the use of S&B Investments or their consultants. The information supplied herein is applicable only for the design of the previously described project to be constructed at location indicated at this site and should not be used for any other structures, locations, or for any other purpose.

We will retain the samples acquired for this project for a period of 30 days subsequent to the submittal date printed on the report. After this period, the samples will be discarded unless otherwise notified by the owner in writing.

FIGURES

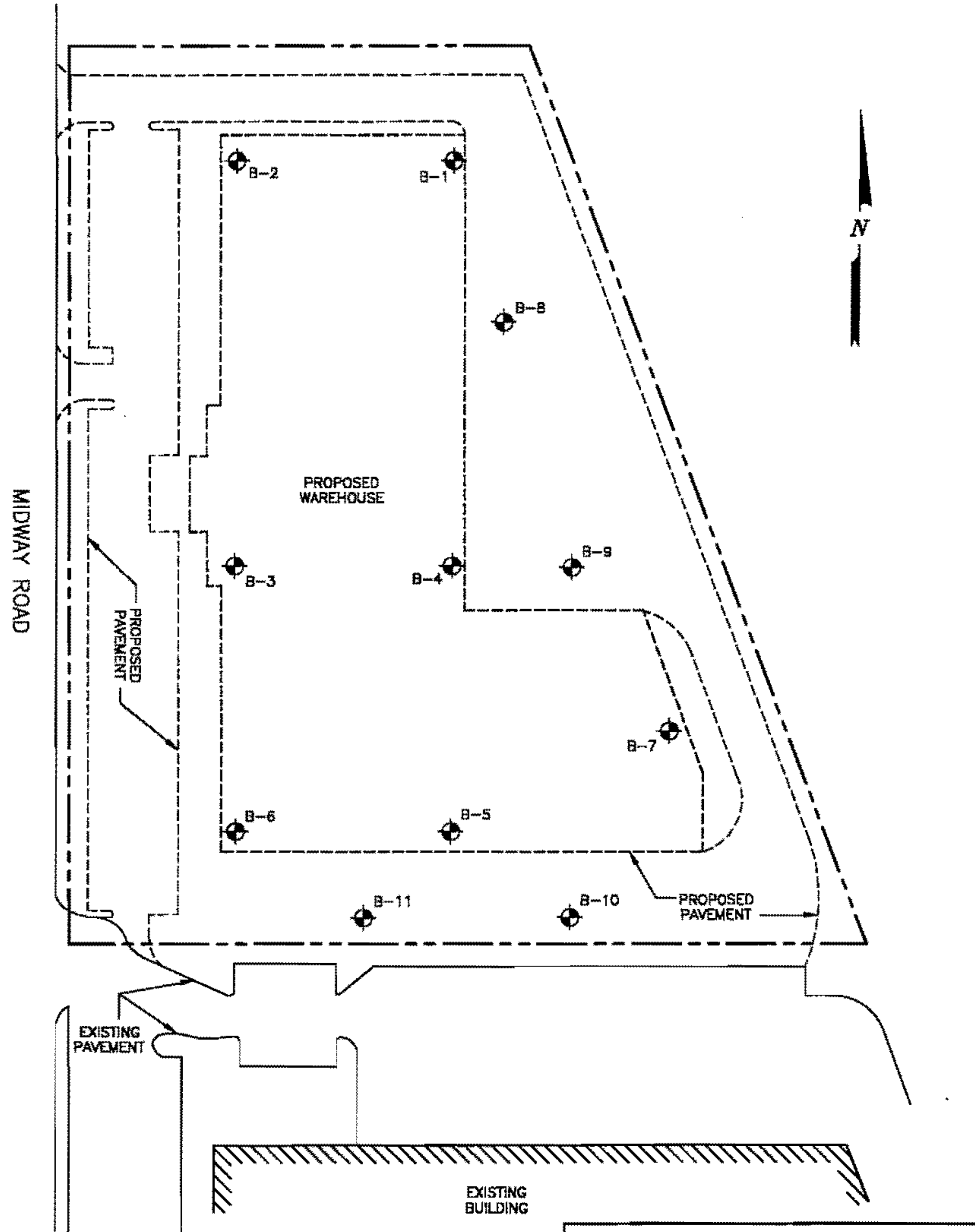


FIGURE 1 BORING LOCATION PLAN

PROPOSED WAREHOUSE–LOT 2/BLOCK 1
BELTWOOD NORTH– AIRPORT ADDITION
ADDISON, TEXAS

DRAWING BY: JD	DATE: 01/31/00	SCALE:
CHECKED BY: DS	DATE: 01/31/00	1"=100'

PROJECT NO.
9912696
FILE NAME:
992696.DWG

MAXIM
TECHNOLOGIES INC

NOTE: BORING LOCATIONS ARE APPROXIMATE.

LOG OF BORING NO. B- 1

PROJECT: BELTWOOD NORTH - AIRPORT ADDITION | SHEET 1 of 1

CLIENT: S&B INVESTMENTS | LOCATION: N. OF 16400 MIDWAY RD. ADDISON, TEXAS

DATE: 1/11/00 | SURFACE ELEV:

DRILLING METHOD: Boring was advanced using air rotary drilling equipment.

GROUNDWATER INFORMATION: Groundwater seepage was encoutered at a depth of 5 feet while drilling. Water at 24 feet at completion of drilling activities.

DEPTH (FT)	N: SPT, BLOWS/FT T: THD, BLOWS/FT P: HAND PEN, TSF	MOISTURE CONTENT, %	DRY DENSITY POUNDS/CU.FT	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	MINUS NO. 200 SIEVE, %	COMPRESSIVE STRENGTH, KSF	FAILURE STRAIN (%)	ABSORPTION SWELL (%)	DESCRIPTION OF STRATUM
0–5.0	P=3.25	22	90.0								Brown CLAY with limestone fragments
	P=4.5+	19	105.0	50	23	27					
5.0–13.0	P=1.75	22									Yellowish brown and light gray clay with calcareous nodules
		23		55	23	32					
13.0–22.0	T=100/										Tan LIMESTONE
	T=100/1"										
22.0–25.0	T=100/.75"										Gray LIMESTONE
25.0											End of Boring at 25'

TUBE SAMPLE | AUGER SAMPLE | SPLIT-SPOON | ROCK CORE | THD CONE PEN. | NO RECOVERY

REMARKS:

9912696

FIGURE 2

MAXIM

LOG OF BORING NO. B- 2

PROJECT: BELTWOOD NORTH - AIRPORT ADDITION | **SHEET** 1 of 1

CLIENT: S&B INVESTMENTS | **LOCATION:** N. OF 16400 MIDWAY RD. ADDISON, TEXAS

DATE: 1/11/00 | **SURFACE ELEV:**

DRILLING METHOD: Boring was advanced using air rotary drilling equipment.

GROUNDWATER INFORMATION: No groundwater seepage was encountered while drilling. Boring was dry at completion of drilling activities.

FIELD DATA				LABORATORY DATA									
SOIL & ROCK SYMBOL	DEPTH (FT)	SAMPLE TYPE	N: SPT, BLOWS/FT T: THD, BLOWS/FT P: HAND PEN, TSF	MOISTURE CONTENT, %	DRY DENSITY POUNDS/CU.FT	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	MINUS NO. 200 SIEVE, %	COMPRESSIVE STRENGTH, KSF	FAILURE STRAIN (%)	ABSORPTION SWELL (%)	DESCRIPTION OF STRATUM
	0–1.0												Brown CLAY with Limestone fragments (FILL) 1.0
			P=3.0	31	92.0	69	26	43					Light gray and yellowish brown CLAY with calcareous nodules
			P=4.5+	23									
	5												
	10		P=3.0	23		57	22	35					13.0
	15		T=100/1.5"										Tan LIMESTONE 18.0
	20		T=100/1.25"										Gray LIMESTONE
	25		T=100/1.25'										25.0
	25												End of Boring at 25
	30												
	35												
	40												

TUBE SAMPLE	AUGER SAMPLE	SPLIT-SPOON	ROCK CORE	THD CONE PEN.	NO RECOVERY

REMARKS:

LOG OF BORING NO. B- 3

PROJECT: BELTWOOD NORTH - AIRPORT ADDITION

SHEET 1 of 1

CLIENT: S&B INVESTMENTS

LOCATION: N. OF 16400 MIDWAY RD. ADDISON, TEXAS

DATE: 1/11/00

SURFACE ELEV:

DRILLING METHOD: Boring was advanced using air rotary drilling equipment.

GROUNDWATER INFORMATION: Groundwater seepage was encoutered at a depth of 14 feet while drilling. Water at 24 feet at completion of drilling activities.

DEPTH (FT)	N: SPT, BLOWS/FT; T: THD, BLOWS/FT; P: HAND PEN, TSF	MOISTURE CONTENT, %	DRY DENSITY POUNDS/CU.FT	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	MINUS NO. 200 SIEVE, %	COMPRESSIVE STRENGTH, KSF	FAILURE STRAIN (%)	ABSORPTION SWELL (%)	DESCRIPTION OF STRATUM
0–2.0	P=4.25	23	105.0								Brown CLAY with limestone fragments and sand (FILL) 2.0
2.0–8.0	P=4.5+	25		75	28	47					Dark brown CLAY with calcareous nodules 8.0
8.0–14.0	P=3.25	24		59	23	36					Light gray and yellowish brown CLAT with calcareous nodules 14.0
14.0–18.0	T=100/1"										Tan LIMESTONE 18.0
18.0–25.0	T=100/.75"; T=100/.5"										Gray LIMESTONE 25.0
											End of Boring at 25'

REMARKS:

TUBE SAMPLE	AUGER SAMPLE	SPLIT-SPOON	ROCK CORE	THD CONE PEN.	NO RECOVERY

LOG OF BORING NO. B- 4

PROJECT: BELTWOOD NORTH - AIRPORT ADDITION

SHEET 1 of 1

CLIENT: S&B INVESTMENTS

LOCATION: N. OF 16400 MIDWAY RD. ADDISON, TEXAS

DATE: 1/11/00

SURFACE ELEV:

DRILLING METHOD: Boring was advanced using air rotary drilling equipment.

GROUNDWATER INFORMATION: Groundwater seepage was encoutered at a depth of 12 feet while drilling. Water at 18 feet at completion of drilling activities.

FIELD DATA				LABORATORY DATA									
SOIL & ROCK SYMBOL	DEPTH (FT)	SAMPLE TYPE	N: SPT, BLOWS/FT T: THD, BLOWS/FT P: HAND PEN, TSF	MOISTURE CONTENT, %	DRY DENSITY POUNDS/CU.FT	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	MINUS NO. 200 SIEVE, %	COMPRESSIVE STRENGTH, KSF	FAILURE STRAIN (%)	ABSORPTION SWELL (%)	DESCRIPTION OF STRATUM
			P=3.25	21									Brown CLAY with limestone fragments (FILL) 1.0
	5		P=4.5+	18		62	24	38					1'-3' No sample - Concrete boulder Brown CLAY with calcareous nodules (possible FILL) 5.0
			P=3.25	25									Brown CLAY with calcareous nodules 8.0
	10, 15		T=100/1.25' ▽ T=100/1"										Tan LIMESTONE 16.0
	20		▼ T=100/.5"										Gray LIMESTONE 20.0
	25, 30, 35, 40												End of Boring at 20'

TUBE SAMPLE	AUGER SAMPLE	SPLIT-SPOON	ROCK CORE	THD CONE PEN.	NO RECOVERY

REMARKS:

LOG OF BORING NO. B- 5

PROJECT: BELTWOOD NORTH - AIRPORT ADDITION

SHEET 1 of 1

CLIENT: S&B INVESTMENTS

LOCATION: N. OF 16400 MIDWAY RD. ADDISON, TEXAS

DATE: 1/11/00

SURFACE ELEV:

DRILLING METHOD: Boring was advanced using air rotary drilling equipment.

GROUNDWATER INFORMATION: Groundwater seepage was encoutered at a depth of 8 feet while drilling. Water at 19 feet at completion of drilling activities.

FIELD DATA				LABORATORY DATA									
SOIL & ROCK SYMBOL	DEPTH (FT)	SAMPLE TYPE	N: SPT, BLOWS/FT T: THD, BLOWS/FT P: HAND PEN, TSF	MOISTURE CONTENT, %	DRY DENSITY POUNDS/CU.FT	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	MINUS NO. 200 SIEVE, %	COMPRESSIVE STRENGTH, KSF	FAILURE STRAIN (%)	ABSORPTION SWELL (%)	DESCRIPTION OF STRATUM
		Tube	P=4.5+	17	106.0								Brown CLAY with limestone fragments (FILL) 3.0
	5	Tube	P=4.5+	20		72	25	47					Brown CLAY with calcareous nodules 8.0
	10, 15	THD	T=100/1.25'; T=100/1.25'										Tan LIMESTONE 18.0
	20	THD	T=100/.75"										Gray LIMESTONE 20.0
													End of Boring at 20'

REMARKS:

TUBE SAMPLE	AUGER SAMPLE	SPLIT-SPOON	ROCK CORE	THD CONE PEN.	NO RECOVERY

LOG OF BORING NO. B- 6

PROJECT: BELTWOOD NORTH - AIRPORT ADDITION

SHEET 1 of 1

CLIENT: S&B INVESTMENTS

LOCATION: N. OF 16400 MIDWAY RD. ADDISON, TEXAS

DATE: 1/11/00

SURFACE ELEV:

DRILLING METHOD: Boring was advanced using air rotary drilling equipment.

GROUNDWATER INFORMATION: No groundwater seepage was encountered while drilling. Boring was dry at completion of drilling activities.

FIELD DATA				LABORATORY DATA									
SOIL & ROCK SYMBOL	DEPTH (FT)	SAMPLE TYPE	N: SPT, BLOWS/FT T: THD, BLOWS/FT P: HAND PEN, TSF	MOISTURE CONTENT, %	DRY DENSITY POUNDS/CU.FT	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	MINUS NO. 200 SIEVE, %	COMPRESSIVE STRENGTH, KSF	FAILURE STRAIN (%)	ABSORPTION SWELL (%)	DESCRIPTION OF STRATUM
			P=3.25	19	103.0								Brown CLAY with limestone fragments (FILL) 2.0
													CONCRETE (FILL) 4.0
	5		P=4.0	32		75	27	48					Brown CLAY with calcareous nodules
	10		P=1.75	32									14.0
	15		T=100/1.75"										Tan LIMESTONE 18.0
	20		T=100/.75"										Gray LIMESTONE
	25		T=100/.25"										25.0
													End of Boring at 25'
	30												
	35												
	40												

TUBE SAMPLE	AUGER SAMPLE	SPLIT-SPOON	ROCK CORE	THD CONE PEN.	NO RECOVERY

REMARKS:

LOG OF BORING NO. B- 7

PROJECT: BELTWOOD NORTH - AIRPORT ADDITION

SHEET 1 of 1

CLIENT: S&B INVESTMENTS

LOCATION: N. OF 16400 MIDWAY RD. ADDISON, TEXAS

DATE: 1/11/00

SURFACE ELEV:

DRILLING METHOD: Boring was advanced using air rotary drilling equipment.

GROUNDWATER INFORMATION: No groundwater seepage was encountered while drilling. Boring was dry at completion of drilling activities.

FIELD DATA				LABORATORY DATA									
SOIL & ROCK SYMBOL	DEPTH (FT)	SAMPLE TYPE	N: SPT, BLOWS/FT T: THD, BLOWS/FT P: HAND PEN, TSF	MOISTURE CONTENT, %	DRY DENSITY POUNDS/CU.FT	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	MINUS NO. 200 SIEVE, %	COMPRESSIVE STRENGTH, KSF	FAILURE STRAIN (%)	ABSORPTION SWELL (%)	DESCRIPTION OF STRATUM
			P=3.0	23		60	25	35					Brown CLAY with limestone (FILL)
			N=47										
	5		P=4.5+	17									
			P=4.5+	15									7.0
													Tan LIMESTONE
	10		T=100/1"										11.0
													Gray LIMESTONE
	15		T=100/.75"										
	20		T=100/.75"										20.0
													End of Boring at 20'
	25												
	30												
	35												
	40												

TUBE SAMPLE	AUGER SAMPLE	SPLIT-SPOON	ROCK CORE	THD CONE PEN.	NO RECOVERY

REMARKS:

LOG OF BORING NO. B- 8

PROJECT: BELTWOOD NORTH - AIRPORT ADDITION

SHEET 1 of 1

CLIENT: S&B INVESTMENTS

LOCATION: N. OF 16400 MIDWAY RD. ADDISON, TEXAS

DATE: 1/11/00

SURFACE ELEV:

DRILLING METHOD: Boring was advanced using air rotary drilling equipment.

GROUNDWATER INFORMATION: No groundwater seepage was encountered while drilling. Boring was dry at completion of drilling activities.

FIELD DATA				LABORATORY DATA									
SOIL & ROCK SYMBOL	DEPTH (FT)	SAMPLE TYPE	N: SPT, BLOWS/FT T: THD, BLOWS/FT P: HAND PEN, TSF	MOISTURE CONTENT, %	DRY DENSITY POUNDS/CU.FT	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	MINUS NO. 200 SIEVE, %	COMPRESSIVE STRENGTH, KSF	FAILURE STRAIN (%)	ABSORPTION SWELL (%)	DESCRIPTION OF STRATUM
			P=4.0	19	108.0								Brown CLAY with limestone and gravel (FILL)
			P=4.5+	10		52	22	30					
	5												5.0 End of Boring at 5'
	10												
	15												
	20												
	25												
	30												
	35												
	40												

TUBE SAMPLE	AUGER SAMPLE	SPLIT-SPOON	ROCK CORE	THD CONE PEN.	NO RECOVERY

REMARKS:

LOG OF BORING NO. B- 9

PROJECT: BELTWOOD NORTH - AIRPORT ADDITION

SHEET 1 of 1

CLIENT: S&B INVESTMENTS

LOCATION: N. OF 16400 MIDWAY RD. ADDISON, TEXAS

DATE: 1/11/00

SURFACE ELEV:

DRILLING METHOD: Boring was advanced using air rotary drilling equipment.

GROUNDWATER INFORMATION: No groundwater seepage was encountered while drilling. Boring was dry at completion of drilling activities.

FIELD DATA				LABORATORY DATA									
SOIL & ROCK SYMBOL	DEPTH (FT)	SAMPLE TYPE	N: SPT, BLOWS/FT T: THD, BLOWS/FT P: HAND PEN, TSF	MOISTURE CONTENT, %	DRY DENSITY POUNDS/CU.FT	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	MINUS NO. 200 SIEVE, %	COMPRESSIVE STRENGTH, KSF	FAILURE STRAIN (%)	ABSORPTION SWELL (%)	DESCRIPTION OF STRATUM
			P=4.5+	17									Brown and dark brown CLAY with limestone fragments (FILL)
			P=4.5+	13	105.0	62	24	38					
	5		P=4.5+	16									5.0
													End of Boring at 5'
	10												
	15												
	20												
	25												
	30												
	35												
	40												

TUBE SAMPLE	AUGER SAMPLE	SPLIT-SPOON	ROCK CORE	THD CONE PEN.	NO RECOVERY

REMARKS:

LOG OF BORING NO. B-10

PROJECT: BELTWOOD NORTH - AIRPORT ADDITION

SHEET 1 of 1

CLIENT: S&B INVESTMENTS

LOCATION: N. OF 16400 MIDWAY RD. ADDISON, TEXAS

DATE: 1/11/00

SURFACE ELEV:

DRILLING METHOD: Boring was advanced using air rotary drilling equipment.

GROUNDWATER INFORMATION: No groundwater seepage was encountered while drilling. Boring was dry at completion of drilling activities.

FIELD DATA				LABORATORY DATA										
SOIL & ROCK SYMBOL	DEPTH (FT)	SAMPLE TYPE	N: SPT, BLOWS/FT T: THD, BLOWS/FT P: HAND PEN, TSF	MOISTURE CONTENT, %	DRY DENSITY POUNDS/CU.FT	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	MINUS NO. 200 SIEVE, %	COMPRESSIVE STRENGTH, KSF	FAILURE STRAIN (%)	ABSORPTION SWELL (%)	DESCRIPTION OF STRATUM	
			P=3.0	25	96.0								Brown and tan CLAY with limestone fragments (FILL)	
			P=4.5+	13		64	25	39						4.0
	5												Dark brown CLAY	5.0
													End of Boring at 5'	
	10													
	15													
	20													
	25													
	30													
	35													
	40													

TUBE SAMPLE	AUGER SAMPLE	SPLIT-SPOON	ROCK CORE	THD CONE PEN.	NO RECOVERY

REMARKS:

LOG OF BORING NO. B-11

PROJECT: BELTWOOD NORTH - AIRPORT ADDITION | SHEET 1 of 1

CLIENT: S&B INVESTMENTS | LOCATION: N. OF 16400 MIDWAY RD. ADDISON, TEXAS

DATE: 1/11/00 | SURFACE ELEV:

DRILLING METHOD: Boring was advanced using air rotary drilling equipment.

GROUNDWATER INFORMATION: No groundwater seepage was encountered while drilling. Boring was dry at completion of drilling activities.

FIELD DATA				LABORATORY DATA									
SOIL & ROCK SYMBOL	DEPTH (FT)	SAMPLE TYPE	N: SPT, BLOWS/FT T: THD, BLOWS/FT P: HAND PEN, TSF	MOISTURE CONTENT, %	DRY DENSITY POUNDS/CU.FT	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	MINUS NO. 200 SIEVE, %	COMPRESSIVE STRENGTH, KSF	FAILURE STRAIN (%)	ABSORPTION SWELL (%)	DESCRIPTION OF STRATUM
			P=2.5	20	99.0								Brown CLAY with limestone & asphalt (FILL)
			P=4.5	16		53	22	31					3.0
													Dark brown CLAY with calcareous nodules
	5		P=4.5+	23									5.0
													End of Boring at 5'
	10												
	15												
	20												
	25												
	30												
	35												
	40												

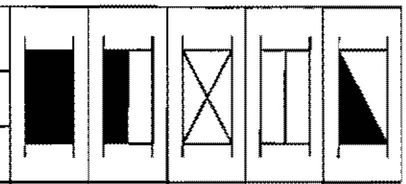
TUBE SAMPLE	AUGER SAMPLE	SPLIT-SPOON	ROCK CORE	THD CONE PEN.	NO RECOVERY

REMARKS:

Symbols and Terms Used on Boring Logs

Soil or Rock Types

	GRAVEL		ORGANIC		SANDSTONE
	SAND		SANDY		SHALE
	SILT		SILTY		LIMESTONE
	CLAY		CLAYEY		FILL



Shelby Tube	Auger	Split Spoon	Rock Core	T.H.D.

Soil Properties

Consistency of Cohesive Soils

DESCRIPTIVE TERM	UNCONFINED COMPRESSIVE STRENGTH (TON/SQ.FOOT)
Very Soft	Less than 0.25
Soft	0.25–0.50
Firm	0.50–1.00
Stiff	1.00–2.00
Very Stiff	2.00–4.00
Hard	More than 4.00

Relative Density of Cohesionless Soils

STD. PENETRATION RESISTANCE BLOWS/FOOT	DESCRIPTIVE TERM
0–10	Loose
10–30	Medium Dense
30–50	Dense
OVER 50	Very Dense

Soil Structure / Description

Soil Structure	Description
CALCAREOUS	Containing deposits of calcium carbonate: generally nodular
SLICKENSIDED	Having inclined planes of weakness that are slick and glossy in appearance.
LAMINATED	Composed of thin layers of varying color and texture.
FISSURED	Containing shrinkage cracks frequently filled with fine sand or silt. Usually more or less vertical.
INTERBEDDED	Composed of alternate layers of different soil types.

Subsurface Water Symbols

▽	WATER LEVEL AT COMPLETION OF DRILLING
▼	DELAYED WATER LEVEL READING

Rock Properties

Relative Hardness of Rock	Description
VERY SOFT OR PLASTIC	Can be remolded in hand: Corresponds in consistency up to very stiff in soils.
SOFT	Can be scratched with fingernail.
MODERATELY HARD	Can be scratched easily with knife: Cannot be scratched with fingernail.
HARD	Difficult to scratch with knife.
VERY HARD	Cannot be scratched with knife.

Description	Grades of Rock
UNWEATHERED	Rock in its natural state before being exposed to atmospheric agents.
SLIGHTLY WEATHERED	Noted predominatly by color changes with no disintergrated zones.
WEATHERED	Complete color change with zones of slightly decomposed rock.
EXTREMELY WEATHERED	Complete color change with consistency, texture, and general appearance approching soil.

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SOIL CLASSIFICATION SYSTEM

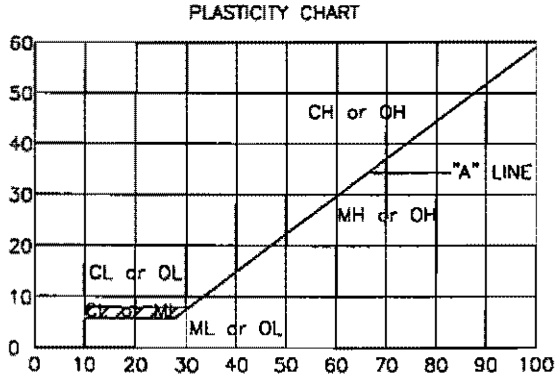
Soils are visually classified according to ASTM D2488 if classification tests are not performed. Grain-size analysis and Atterberg Limit Tests are often performed on selected samples to aid in classification. The classification system is briefly outlined on this chart. For a more detailed description of the system, please refer to ASTM Designation D-2487.

MAJOR DIVISIONS			SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of material is LARGER than No. 200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 sieve size)	CLEAN GRAVELS (Little or no fines)	GW	Well graded gravels, gravel - sand mixtures, little or no fines
			GP	Poorly graded gravels or gravel - sand mixtures, little or no fines
		CLEAN GRAVELS (Appreciable amt. of fines)	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS (More than 50% of coarse fraction is SMALLER than the No. 4 sieve size)	CLEAN SANDS (Little or no fines)	SW	Well graded sands, gravelly sands, little or no fines
			SP	Poorly graded sands or gravelly sands, little or no fines
		SANDS WITH FINES (Appreciable amt. of fines)	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS (More than 50% of material is SMALLER than No. 200 sieve size)	SILTS AND CLAYS (Liquid limit GREATER than 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 AND CLAYS (Liquid limit LESS than 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 silts.
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils.

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

NOTE: COARSE SOILS WITH BETWEEN 5% & 12% PASSING THE NO. 200 SIEVE AND FINE GRAINED SOILS WITH LIMITS PLOTTING IN THE HATCHED ZONE OF THE PLASTICITY CHART ARE TO HAVE DUAL SYMBOLS

PLASTICITY CHART



DEGREES OF PLASTICITY OF COHESIVE SOIL *

DEGREE OF PLASTICITY	PLASTICITY INDEX
NONE	0 - 4
SLIGHT	5 - 10
MEDIUM	11 - 20
HIGH	21 - 40
VERY HIGH	>40

* AFTER BURMISTER (1948)

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APPENDIX A

Measures To Reduce the Risk of Free Water Sources

APPENDIX A
MEASURES TO REDUCE THE RISK OF FREE WATER SOURCES

In order to reduce the risk of excessive upward ground movements caused by soil swelling associated with free water sources, the following measures should be taken during design and construction:

- The use of superior utility contractors and utility line materials accompanied with Quality Control inspection and testing of all utility line installations including automatic sprinkler systems installed after construction.

- Utility under-drains with impervious barriers along the trench bottom may be used as an additional safeguard at lots where it is desired to minimize post-construction upward movement.

- Elevated landscaped beds should be used in lieu of recessed beds to prevent ponding water conditions near the structure.

- Positive drainage should be provided at all lot locations. Lot drainage near structure: 3% minimum. Lot drainage swales: 1% minimum.

- Roof gutters should be used to direct roof runoff away from the structure in the most direct manner. Downspouts should not be allowed to discharge into landscaped areas located near the structure. Downspouts extensions should be used to facilitate rapid drainage away from the structure.

- If retaining walls are required due to site topography, drainage swales, having a minimum 1 percent slope, should be provided near the top of the retaining walls to prevent runoff from the up slope property from draining onto the lower adjacent property.

RESPONSIBILITY OF OWNER

- Use of superior contractors and materials for installation of sprinkler systems and Quality Control inspection and testing of systems installed. Sprinkler lines should not be installed near the structure. Instead, the system should be designed so that the lines themselves are as far away from the structure as possible. Sprinkler heads should be used with a capacity to direct water toward the structure from distances of several feet.

- Rapid repair of any utility leak including water lines, sewer lines, sprinkler lines, sprinkler heads.

- Maintaining site drainage provided by the builder, particularly in landscaped areas adjacent to the structure.

- Using elevated landscaped beds in lieu of recessed planters to prevent ponding water conditions near the structure. Gutter downspout extensions should be added in all areas containing landscaped beds to prevent downspout discharge into the beds.

- Trees and deep rooted shrubs should be located no closer to the structure than one-half their ultimate mature height to reduce foundation settlement effects caused by moisture absorption of the root systems.

- A moist soil condition (not a soaked condition) must be maintained within 5 feet of the foundation during prolonged periods of dry weather to prevent differential settlements caused by ground shrinkage.

APPENDIX B

Specifications for Water Pressure Injection

APPENDIX B
SPECIFICATIONS FOR WATER PRESSURE INJECTION

SITE PREPARATION

Prior to the start of injection stabilization, the building areas should be staked out to accurately mark the area to be injected. The area to be injected should extend at least five feet beyond the limits of the building areas and adjacent sidewalks. Allowance should be made for swelling that may occur as a result of the injection process depending on soil properties and in-situ moisture.

EQUIPMENT AND MATERIALS

1. The injection vehicle shall be capable of forcing injection pipes into the soil with minimal lateral movement to prevent excessive blowback and loss of liquid around the injection pipes. The vehicle may be rubber tire or track mounted suitable for the purpose intended.

2. Slurry pumps shall be capable of pumping at least 3000 gph at 50-200 psi.

3. A nonionic surfactant (wetting agent) shall be used according to manufacturer's recommendations, but in no case shall proportions be less than one part (undiluted) per 3,500 gallons water.

APPLICATION

1. Injection stabilization work shall be accomplished prior to installation of any plumbing, utilities, ditches or foundations.

2. The injection pressures shall be adjusted as directed by a Maxim technician within the range of 50 to 100 psi to inject the greatest quantity of fluid into the soil mass. In order to assure that the pressure is within this specified range, each injection vehicle shall be equipped with an accurate pressure gauge attached to the manifold (the pipe fitting on which the probe valves are attached).

3. Space injection so as not to exceed five feet on center each way, and inject a minimum of five feet outside building areas.

4. Injection shall either proceed from the ground surface downward to the specified depths or in an upward manner beginning at the specified injection depth and proceeding upward, as directed by a Maxim technician. Inject fluid to the required depth, or to impenetrable material, whichever occurs first. Impenetrable material is the maximum depth to which two injection rods can be mechanically pushed into the soil using an injection machine having a minimum gross weight of five tons. Injections are to be made in 12" to 16" intervals, with a minimum of six stops for seven feet and eight stops for ten feet. The probes shall be forced into the soil, not washed down by scouring action of the fluids. The lower portion of the injection pipes shall contain a hole pattern that will uniformly disperse fluid in a 360 radial pattern. Inject at each interval to "refusal" (i.e. until the maximum quantity of fluid has been injected into the soil and fluid is running freely at the surface, either out of previous injection holes or from areas where the surface soils have fractured around each injection probe). Backpressure flow out of previous injection holes shall not constitute "refusal". Fluid coming up around or in the vicinity of one injection probe shall also not be considered as refusal. If this occurs around any probe, this probe shall be cut off so that water can be properly injected through each probe at each 12 inch injection depth interval. If this occurs around any probe, this probe shall be cut off so that water can be properly injected through each probe at each 12 inch injection depth interval. In any event, no probe shall be cut off within the first 30 seconds of injection (after verification of no blockage as specified below). The 30 second criterion is a <u>minimum</u> time for each 12 inch depth interval and <u>not</u> a maximum time limit.

   The injection vehicle shall be fitted with individual cut off valves for each probe. At each 12 inch interval, each valve will be cut off and on to assure that each probe is not blocked and that water is flowing. If one or two probes are blocked, the others shall be cut off so that the added pressure will clear out the blockage.

5. After a minimum curing time of 48 hours, the injected pad may be tested to determine if additional injections with water and surfactant are necessary. The water injections will be five feet on center each way and spaced 2½ feet offset in two orthogonal directions from the initial injection.

6. A minimum of 48 hours shall elapse between each injection application in any one area to allow for moisture absorption, if required.

7. After four injection applications, the surface soils shall be scarified and recompacted to form a surface seal prior to additional injections.

8. The required final moisture content shall be controlled by swell test results as outlined below.

9. Upon completion of the final injection, scarify the top eight inches of soil and recompact to a minimum of 93% Standard Proctor density (ASTM D 698), at a moisture content ranging from +3 to +6 percentage points above the optimum moisture value.

OBSERVATION AND TESTING

1. A full-time Maxim engineering technician will be present throughout the entire injection operation. After completion, undisturbed samples will be taken at one foot intervals to the total depth injected as specified by the Geotechnical Engineer.

2. Inspection, test drilling and verification of moisture contents will be performed under the direction of the Geotechnical Engineer.

3. Moisture content tests and hand penetrometer determinations shall be performed on one foot intervals. One dimensional swell tests shall be performed on selected soil samples. The number of swell tests along with the corresponding depths will be selected by the Geotechnical Engineer in such a way that the PVR for each test boring can be estimated. One dimensional swell tests shall be conducted in a manner similar to that of ASTM D 4546-85 Method B.

4. The average swell from each test boring shall not exceed 1.0 percent, and the PVR for a 10 foot depth for each boring shall not exceed one (1) inch. This criteria is based upon a design PVR of one (1) inch within the depth of treatment.

5. Where swell criteria is not met, reinjection will be required. Additional testing will be performed in the reinjected areas.

6. The surface of the injected area should be sealed or otherwise protected against moisture loss.

7. After approval of the injection operations, standing water should be removed and the subgrade be proofrolled. The subgrade should then be excavated to select fill subgrade and compacted to a minimum of 93% of the maximum dry density as determined by ASTM D 698 (Standard Proctor) between +3 and +6 percentage points above the optimum moisture content.

8. The moisture condition of the completed pad must be maintained until all slabs are in place.

12-20-00

Vic Sahm Project / Midway Rd.
Prop. Bldg F.F. El. 623.5

El. Prop Taxiway "B" 630-632 ±

27'-11" door ht accommodates G4
24 clear height proposed for the building