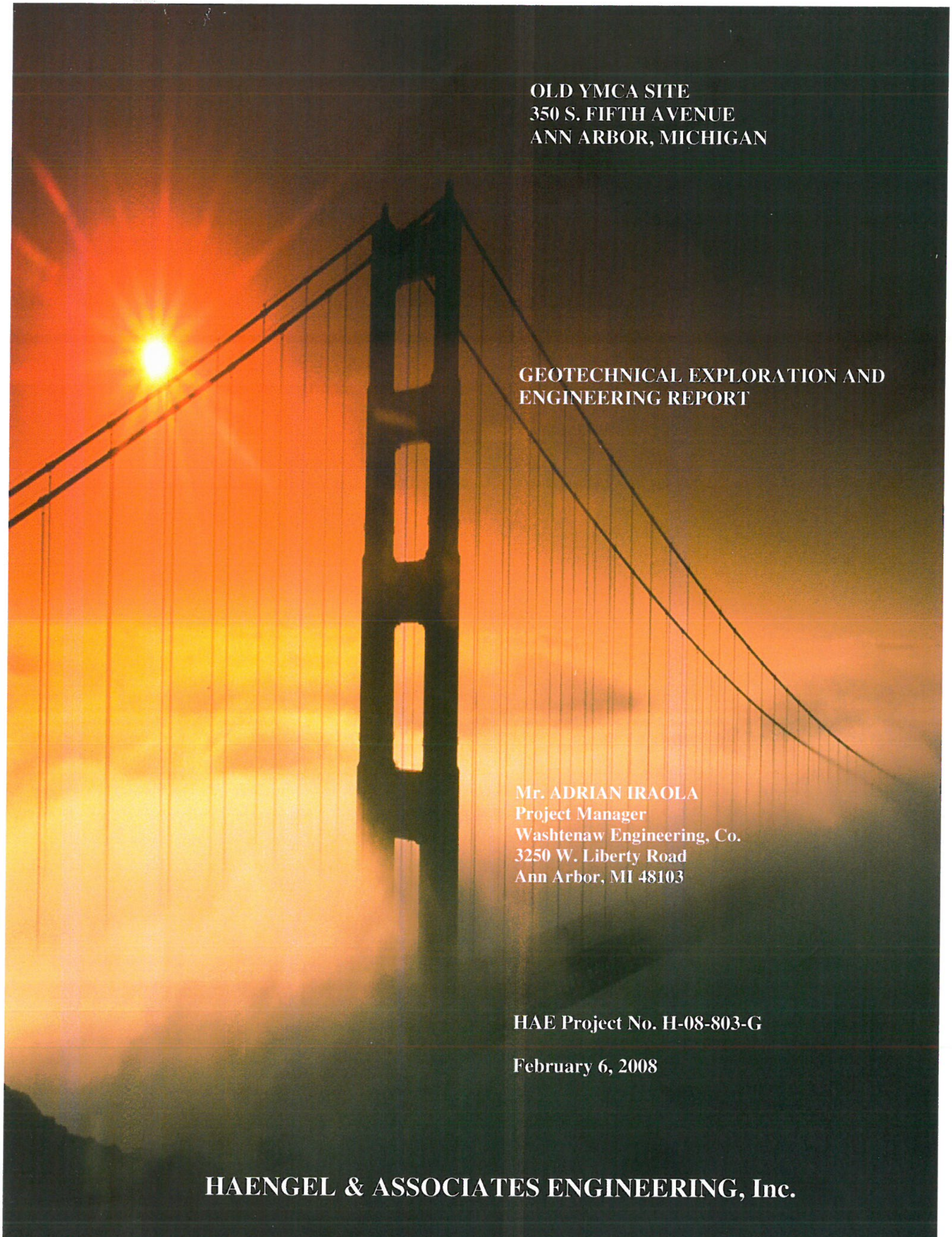

OLD YMCA SITE
350 S. FIFTH AVENUE
ANN ARBOR, MICHIGAN

GEOTECHNICAL EXPLORATION AND ENGINEERING REPORT

Mr. ADRIAN IRAOLA
Project Manager
Washtenaw Engineering, Co.
3250 W. Liberty Road
Ann Arbor, MI 48103

HAE Project No. H-08-803-G

February 6, 2008

HAENGEL & ASSOCIATES ENGINEERING, Inc.

HAE H.A.E., Inc.
ENGINEERING DIVISION
Geotechnical Environmental Structural Construction

42030 KOPPERNICK RD., SUITE 318
CANTON, MICHIGAN 48187
OFFICE: 734-455-9771
FAX: 734-455-9774
E-MAIL: haei@earthlink.net
WEBSITE: www.haengel.com

February 6, 2008

Mr. Adrian Iraola
Project Manager
Washtenaw Engineering Co.
3250 W. Liberty Road
Ann Arbor, Michigan 48103

Re: Geotechnical Exploration and Engineering Report
Proposed Old YMCA Site
350 S. Fifth Ave.
Ann Arbor, Michigan
HAE Project No. H- 08-803-G

Dear Mr. Iraola:

We and our partner company Inspec Sol have completed the geotechnical exploration and engineering report for the proposed First and William Street Parking Structure project, located in Ann Arbor, Michigan. This report presents the results of our observations and analysis and our recommendations for subgrade preparation, and construction considerations. The purpose of this study was to obtain general subsurface information from the site and provide recommendations for the subsoil conditions for the proposed parking structure. Enclosed are copies of the logs of the 3 geotechnical borings drilled to evaluate soil and groundwater conditions at the site, as well as the geotechnical recommendations for design and construction of this project.

As part of the testing program, HAE recommends that a test pile program be implemented for the pile foundations, including dynamic and static load testing.

The purpose of dynamic testing is to monitor pile compression and tension driving stresses, check pile integrity, assess driving system performance, and evaluate the development of pile capacity with respect to penetration and depth. Prior to dynamic testing, a wave equation analysis using computer models such as GRLWEP by Pile Dynamics, Inc. should be completed to establish preliminary driving criteria. The wave equation analysis should be based on the pile type and size, and the intended pile driving equipment proposed and submitted by the contractor.

A dynamic testing program should consist of a minimum of tree indicator piles. The indicator piles should be located throughout the site and at representative loading conditions based on final building design and loads. The dynamic testing program should also include a program in which the indicator piles have a re-strike test after a 7-day waiting period after installation. This is to evaluated conditions of either soil relaxation or soil set-up (soil "freeze") that occurs due to the vibrations in the foundation soil during driving operations.

Based on the dynamic test data acquired during initial driving and re-strike testing of the indicators piles, two indicator piles should be selected for subsequent static load testing. The test piles should be loaded to 2 times the allowable design load.

Upon completion of the indicator pile testing program, a summary report of test results and production pile installation criteria should be provided. This report should include presentations of the dynamic testing results versus pile penetration depth. The Correlation between static load test and dynamic tests should be also evaluated, and CAPWAP analyses of the dynamic testing data should be performed to refine the production pile installation criteria.

We appreciate the opportunity to assist you and the design team on this project. If you have any questions regarding this report, please do not hesitate to contact us.

Thank you very much for your continued use of our services.

Respectfully,

HAENGEL & ASSOCIATES ENGINEERING, INC.

Gustavo N. Haengel

Gustavo N. Haengel
Principal
encl:
3 pc: encl.

INSPECSOL ENGINEERING INC.
MEMBER OF THE CRA FAMILY OF COMPANIES

14496 Sheldon Rd., Suite 200, Plymouth, Michigan 48170 • Tel.: (734) 453-5123 • Fax: (734) 453-5201

Reference No. D020036

February 4, 2008

Mr. Gus Haengel
Haengel and Associates Engineering, Inc.
42030 Koppernick Road
Suite 318
Canton, Michigan 48187

Dear Mr. Haengel:

Re: Preliminary Geotechnical Engineering Evaluation
Old YMCA Site
Ann Arbor, Michigan

Inspecsol Engineering Inc. (Inspecsol) is pleased to submit to Haengel and Associates Engineering, Inc. the enclosed Preliminary Geotechnical Engineering Evaluation Report for the Old YMCA site in Ann Arbor, Michigan. Two bound copies are provided for your use. The report presents the preliminary recommendations for foundation design and general construction related issues.

We trust that this report is to your satisfaction. We appreciate providing our services to you, and look forward to continuing these services during the final geotechnical engineering phase of the site development. Should you have any questions, contact us at (734) 453-5123.

Respectfully Submitted,

INSPECSOL ENGINEERING INC.

Michael C. Gentner, P.E.
Vice President

MCG/ds

INSPEC-SOL

Reference No. D020036

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(Following Text)

LIST OF APPENDICES

1.0 INTRODUCTION

Inspecsol Engineering Inc. (Inspecsol) was retained by Haengel and Associates Engineering, Inc. (HAE) to complete a preliminary geotechnical evaluation for a proposed mixed-use, mid-rise structure. The planned construction and future use of this site is unknown at this time. The existing building will be demolished. Preliminary discussions are to replace it with an 8 to 10 story building and potentially have up to 3 sub-levels of parking. The site is located at the northeast corner of 4th Avenue and East William Street in Ann Arbor, Michigan. A Site Location Diagram is included in the appendix as Figure 1.

The site currently consists of a multi-level building with surface parking lot. The building was formerly home to the Ann Arbor YMCA organization. The site is bounded by 5th Avenue to the east, 4th Avenue to the west, William Street to the south, and the Ann Arbor Transit Authority depot to the north.

The purpose of the evaluation was to obtain preliminary information regarding subsurface soil and groundwater conditions to aid in initial development planning. Specifically, our scope was to provide evaluation of the soils to evaluate allowable soil bearing capacities, to evaluate the seismic Site Class per current Michigan Building Code, and to provide recommendations for new foundations. Geotechnical recommendations are being provided with respect to allowable bearing pressures, and suitable foundation bearing strata. Additionally general recommendations regarding earth-retention options for below grade construction and general construction will be given. Our specific scope of work authorized by HAE was:

- Drill six (3) borings at locations within the site property to depths ranging from 70 to 100 feet; and
- Provide recommendations for foundation design and construction considerations for the proposed building structure.

2.0 SOIL AND GROUNDWATER CONDITIONS DATA

The field exploration was conducted December 17, 2007 through December 21, 2007, and consisted of drilling 3 boreholes (B-1, B-2 and B-3) to depths ranging from 30 to 100 feet below existing site grades. Boreholes B-1 and B-2 were drilled near the north and south sides of the existing building's east canopy, respectively. Borehole B-3 was drilled near the southwest property corner. The borehole locations are shown on the enclosed Borehole Location Plan as Figure 2. The borehole logs are attached in Appendix A.

American Drilling and Testing Company completed the drilling under the full-time supervision of Inspecsol geotechnical staff. Borehole B-1 was drilled to 30 feet before a boulder obstruction caved into the borehole at 20 feet, and subsequently could not be re-drilled through. Borehole B-1 was abandoned at 30 feet. Boreholes B-2 and B-3 were drilled to 100-foot depths below existing grade. The boreholes were drilled with a truck-mounted CME-75 drill rig using 4-1/4" ID hollow stem augers to approximately 15 feet. Wash rotary methods were used to advance the boreholes to final depths, using 13 to 22 feet of temporary steel casing to prevent fluid loss and borehole caving. Representative soil samples were obtained at 2.5-foot intervals to a depth of 10 feet below existing grade, and at 5-foot intervals thereafter by using a 2-inch diameter split spoon barrel sampler in general accordance with ASTM D-1586. At some sample intervals, a 3-inch long brass liner insert was used to collect samples for unit weight determination. These select intervals are noted on the enclosed borehole logs. Penetration resistance, measured in blows per foot (bpf), was recorded as N-values on the borehole logs.

The borehole locations were selected and located in the field by HAE with marking paint. Using a temporary benchmark with assumed elevation of 100.00 feet, ground surface elevations at the borehole locations were measured and range from 94.0 to 99.6 feet. The west side bottom flange of the northwest corner fire hydrant at 5th Street and William Street was used for the benchmark. Groundwater observations were made in the boreholes during drilling and sampling, and upon completion of all borehole tests. The boreholes were backfilled with a Portland cement/bentonite grout, and capped with topsoil/drill cuttings upon completion.

Soil samples obtained from the boreholes were field classified upon retrieval for type, texture, color and moisture condition. The samples were sealed in clean, airtight, glass containers and plastic bags. All samples were transported to our geotechnical laboratory in Plymouth, Michigan for further examination and testing.

2.1 *Laboratory Testing*

All samples received in the lab were visually examined by an experienced geotechnical engineer, and classified on the basis of type, texture, plasticity, color, relative density and consistency in general accordance with the Unified Soil Classification System. Moisture content determination, unit weight measurements, and grain size analyses were completed on select samples to further determine soil index parameters for classification.

3.0 SITE AND SUBSURFACE CONDITIONS

The following presents a summary of the subsurface conditions at the site based on the data obtained from the investigation. Details of the subsurface conditions encountered at the site are provided on the Borehole Logs presented in Appendix A. It should be noted that the subsurface conditions are based on those encountered at the specific borehole locations. These conditions may vary at other locations of the site both horizontally and vertically. The boundaries between various strata, as shown on the boring logs, are estimated in some cases. These boundaries represent an inferred transition between the various strata, rather than a precise plane of geologic change.

3.1 Site Geology

Based on the Hydrogeologic Atlas of Michigan by Western Michigan University (1981), the geology of the area consists of glacial drift soils that may range from 200 to 400 feet in thickness. The overburden soils are characterized by outwash and glacial channels, moraines, and ground moraines (till plains). The overburden soils are underlain by Mississippian-aged bedrock that includes the Coldwater shale formation.

3.2 Soil Conditions

Based on review of the soil conditions presented in the borehole logs, the following generalized subsurface profile is given. The depths are referenced to the ground surface elevation at the time of completion.

Fill Soil: Fill soil was encountered at the surface in boreholes B-1 and B-2 and extended to approximately 3 to 5.5 feet in thickness. The fill soils consist of mixed very loose to loose sands with varying amounts of gravel.

Compact to Very Dense Sand: Below the upper fill deposit (and from the ground surface at B-3) is predominately fine to coarse, compact to dense poorly graded sand (SP and SP-SM). The sands contain varying amounts of silt and gravel, with areas of cobbles and/or boulders generally identified between 15 to 30 feet based on the observations of the drill rig response (hard drilling and rig chatter). The depth at which this granular strata extends to is to the termination of the boreholes at 100 feet below the existing ground surface at borehole locations B-1 and B-2, and to 87 feet at borehole B-3. The Standard Penetration Test (SPT) N-value for the sand ranges from 20 to 76 (bpf) with an average of 30 bpf. In general, the relative density of the soils encountered is compact to dense, and the higher blow counts generally tend to coincide with strata containing more coarse sand and gravel.

Compact Clayey Silt: At borehole B-3, compact clayey silt (ML) was encountered below the sand at a depth of 87 feet and extends to the termination depth of the borehole. The SPT N-value for the clayey silt ranges from 18 to 24 blows per foot, with an average of 22 bpf.

3.3 Groundwater Conditions

Groundwater was not encountered in the boreholes during or after drilling operations. The boreholes were advanced with a wash rotary method. This method of drilling introduces water and drilling fluid into the borehole in order to stabilize the sidewalls. This makes it difficult to determine when, and if, groundwater is encountered. Without installing a monitor well or piezometer, the groundwater level can be estimated by the change in color from brown to gray. Based on this estimate, the long-term groundwater is expected to be around 77 feet. We recommend monitor wells or piezometer be installed for more accurate groundwater measurements during the final geotechnical evaluation.

Other perched water levels may be encountered at higher levels than stated above. Seasonal fluctuations in the level of the groundwater table should be expected over the course of the year due to seasonal precipitation and runoff effects.

4.0 DISCUSSION AND RECOMMENDATIONS

The planned construction and future use of this site is unknown at this time. The existing building will be demolished. Preliminary discussions are to replace it with an 8 to 10 story building and potentially have up to 3 sub-levels of parking.

The following general recommendations are based on the results of the field and laboratory testing, and our geotechnical analysis. These recommendations should be considered preliminary for planning purposes only. Once the final building type, location, size, and loads are known, a site-specific geotechnical exploration to augment those presented herein should be completed, as well as detailed engineering analysis.

4.1 Earthwork Operations

4.1.1 Subgrade Preparation

For areas where slabs or pavements may be constructed, the following recommendations apply. These are applicable recommendations for slabs or pavements placed near the existing ground surface, and for slabs that may be placed at the lowest level of any sub-level parking structure or basement. The exposed subgrade should be proof rolled and compacted with a smooth drum roller. Any areas showing deflection or rutting should be re-worked to an appropriate moisture condition, or removed and replaced with engineered fill if deemed unsuitable soil.

Backfill material should consist of non-frozen, non-organic natural soils free of deleterious material or debris. Backfill material should be as uniform as practical and mixing of soils should be avoided. The fill soil should be placed in maximum loose lifts of 8 inches, and compacted to a minimum of 95% of its maximum dry density, as determined by the Modified Proctor test (ASTM D-1557). Conditioning of the soil to achieve the desired moisture content by sprinkling or scarifying and aerating can be expected. The moisture content should be +/- 3% of optimum, as determined in the modified proctor test.

Engineered fill placement and compaction should be under the full time observation of a qualified geotechnical technician. In situ density tests should be taken frequently to measure the specified degree of compaction has been met.

4.2 *Foundation Design*

4.2.1 *Spread Footings*

For areas where basements or below-grade parking are not present, shallow footings on the native compact sand can be used to support relatively light to moderate column and wall loads. For footings placed approximately 5.5 feet below existing grade, a net allowable bearing capacity of 7,000 pounds per square foot (psf) can be used in design. The net allowable bearing capacity is that pressure that can be transmitted to the bearing strata in excess of the final minimum overburden pressure near the footing. The footings should extend through the existing fill soil encountered in the boreholes to a depth up to 5.5 feet. Prior to footing concrete placement, qualified geotechnical personnel should inspect the base of all foundation excavations to confirm the design bearing pressures and material adequacy of the foundation soil. Any localized loose zones present at the footing bearing elevation should be sub-excavated and replaced with engineered fill or lean concrete as directed by the engineer. Crushed concrete or crushed stone could also be placed in the bottom of the footing excavation to improve stability. The net allowable bearing capacities given have a factor of safety of 3.

The footings should be constructed a minimum of 3.5 feet below final exterior grades. We recommend that individual column footings have a minimum width of 2.5 feet, and continuous footings have a minimum width of 1.5 feet.

If basements and/or parking levels are excavated to approximately 40 feet below existing grade, footings placed on the compact to very dense sands could be proportioned and sized for a maximum allowable bearing capacity of 10,000 psf.

4.2.2 *Deep Foundations*

If column loads are such a magnitude that individual spread footings are not practical, deep foundations could be advanced to bear within the underlying dense sand deposits. Four deep foundation types were considered in our preliminary analysis and presented; they are micropiles (or referred as minipiles), augured-cast-in-place (ACIP) piles, driven piles, and drilled piers. Each deep foundation type is discussed in detail in the following sections. The majority of the load carrying capacity of each system is derived from side friction between the soil and structural element. Because of the unknown nature of any structure planned at this site, and the wide array of capacities that can be developed for each system based on diameter and length, we have for comparison purposes limited capacity analyses to a 50-foot depth below existing grade, and assumed no underground levels will be built (deep foundations installed at existing ground level).

For any deep foundation system installed from existing street level, there is the possibility of encountering cobbles and boulders in the upper 20 to 25 feet. This of course will cause installation difficulties for any of the options given herein.

4.2.2.1 Micropiles

The micropile is installed by rotating a drill pipe or casing (usually 5 to 12 inch diameters) into the ground to the desired depth. As the casing is drilled in, the soil inside is flushed out with drilling fluid, usually water or drilling mud. The inside of the casing is tremie grouted to displace the drill fluid, and the reinforcing steel is placed. The drill casing is then pulled out of the ground to expose the desired "grouted bond zone" while additional grout is added under a pressure at the top of the casing. Another option would be to leave the drill casing at full depth and used as part of the reinforcing steel portion of the pile element, and pressure grout so there is grout contact between the outside wall of the casing and the soil within the desired grouted bond zone. The pile derives its load carrying capacity from side friction between the soil and grout along the bond zone.

For preliminary design purposes, a 12-inch diameter micropile with a grouted bond zone length within the sand soil of 50 feet will provide an allowable individual pile capacity of 120 kips. The capacity is based on a bond capacity between the soil and grout of 2,000 psf, and a factor of safety of 2.5 for side friction.

The advantage of micropiles is the small diameters, ability to install through the cobbles and boulders that may be present, and ability to install on a batter if required. The drill casing can be incorporated into the final pile cross-section to comprise the final pile element, and then there may be no need for additional steel reinforcement.

The disadvantage of the micropile is the smaller diameters and development of capacity through side friction only result in needing pile groups within each pier cap. The smaller diameters may also not be able to resist any lateral loads adequately. This can somewhat be countered by installing them at a batter, but will be dependent on the magnitude of the lateral loads, allowed deflection, and available space to install cluster of micropiles in combination to resist both vertical and lateral loads. Also, some of the coarse layers of sand, gravel, cobbles and boulders may have larger than anticipated grout takes, and excessive grout loss may cause the need for the drill casing to remain in place.

It should be recognized that the successful installation of micropiles is highly dependent of the contractor's installation techniques. Therefore, only experienced micropile contractors with demonstrated abilities should be considered. Extreme care must be exercised during the installation and grout placement of the micropiles to make sure a competent grouted bond zone is provided.

4.2.2.2 Auger Cast-in-Place Piles (ACIP)

The ACIP is installed by drilling to a specified depth with a continuous flight hollow stem auger and then pumping cement-based grout (either by tremie, or with combination of pressure grouting) into the drilled hole while carefully extracting the auger and maintaining an adequate head of grout inside the auger to prevent side wall caving. The pile derives its load carrying capacity from both end bearing and side friction along the shaft.

For example, a 24-inch diameter ACIP approximately 50 feet into the sand strata could achieve an allowable compression capacity of 400 kips. A factor of safety equal to 3 for end bearing and 2 for side friction was used. Other allowable capacities can be used based on the ACIP diameter and depth drilled within the sand strata.

It should be recognized that the successful installation of ACIP is highly dependent of the contractor's installation techniques. Therefore, only experienced ACIP contractors with demonstrated abilities should be considered. Extreme care must be exercised during the installation and grout placement of the ACIPs to make sure a continuous shaft of the required diameter is installed.

Some of the problems that can be incurred during installation that can go unobserved or undetected are:

- If the augers are extracted too quickly or if the grout is not pumped continuously or fast enough through the augers, a void or decrease in grout pressure will result in either water and/or soil entering the void ahead of the grout. This result of intrusion of soil and groundwater into the grout, thus disrupting the pile by decreasing the diameter and effective strength, and consequently its structural integrity.

- Mechanical breakdowns during installation, especially during the grout operation, can result in significant decreases in the grouting pressure. Under certain conditions, soil and groundwater can exert greater lateral pressures onto the pile, this causing a decrease in the diameter of the pile.
- While extracting the augers, care must be exercised so that the augers are extracted in a smooth, constant motion. Ideally the grouting pressures should push out the augers. If the mechanical equipment does not allow a slow, constant extraction, the chances of having a disruptive or discontinuous pile will increase.
- The contractors' equipment for the duration of the project has to be in good working condition, with extra monitoring gauges, stand-by pumps, and other accessories available at the site prior to starting.
- Consideration must be given to inadvertent or unanticipated movements that could be transferred to the upper portion of the piles. Therefore, proper control of the location of the piles in relation to the pile cap is important. Reinforcing steel should be provided, as necessary, and the reinforcing steel protruding and tied into the pile cap.

The major disadvantages ACIPs may have for this site is the required torque to advance the augers (especially the larger diameter piles) through the cobbles and boulders present within the upper 20 to 25 feet. The presence of the cobbles and boulders may also cause the augers to drift out of plumb. As mentioned with the micropiles, grout takes may be affected by the coarser gravel and cobble layers, and irregular pile cross-sections may be caused by this.

4.2.2.3 Driven Piles

Piles should be driven to a predetermined driving criteria based on a wave equation analysis. The driving criteria need to consider allowable stresses in the pile while driving, the type, size and energy of the hammer, and the use and type of pile caps and hammer cushions. The use of pile driving shoes will be required. For an HP14x74 driven to 50 feet below existing grade, an allowable capacity in compression of 100 kips can be considered. A factor of safety of 3 for end bearing and 2 for side friction was used.

The advantage of driven piles is the relative ease of installation and daily production. Also, with a driven system, the need for additional equipment such as grout pumps that can disrupt the installation process are not required. The driven system will also provide a better means of determining if the desired capacity is met for each pile by the simple means of recording the number of hammer blows to advance the pile, and comparing it to the aforementioned wave equation analysis.

The disadvantage of the driven pile is the vibrations caused during driving and the detrimental effect they can have on adjacent structures. For driven piles, a pre-driving study should be completed to "inventory" nearby structures, their foundation types, and the effects the assumed vibrations from pile driving may have on them. A vibration-monitoring program should also be considered during pile driving, with pre-determined limits of peak particle velocities set. The other obvious disadvantage of driven piles would be refusal prior to reaching the design tip elevation due to cobbles and boulders. This can also cause high driving stresses within the pile elements that can damage them during installation.

4.2.2.4 Drilled Shafts

For a 4-foot diameter straight shaft drilled pier at 50 feet below existing grade, an allowable capacity of 1,000 kips can be considered. The capacity considers both end bearing of the shaft and side friction between the shaft walls and the soil.

Because of the relatively dry sands with very little fines, they create a "running sand" condition, and as such sidewall stability of drilled shafts are a concern. The shafts will need to be advanced full depth with temporary steel casing, or with a polymer based drilling fluid used. At a depth of 50 feet, the bottom of the shaft should be above any significant water table. If full-depth temporary casing is used, and the inside drilled out, concrete could be placed in the dry by the free fall method from the ground surface. If drilling slurry is used to advance the drilled shafts, concrete would need to be placed by tremie method from the bottom up to displace the slurry.

The advantages of drilled shafts is the higher capacity per pier compared to the other pile elements, and the ability to provide higher lateral load resistance. The major disadvantage is advancing the larger diameter through the cobble and boulder zones as compared to the different pile elements previously discussed.

4.3 Seismic Considerations

Evaluation of seismic classification was based on the SPT data and undrained shear strength estimates reported in the borehole logs. The soil profile consists of cohesionless soils with weighted average "N" values less than 50 blows per foot (48.5 with current data). Therefore, based on information in the 2003 Michigan Building Code, the Site Class D should be used for preliminary seismic design. Other borings drilled to 100 feet during the final geotechnical evaluation may yield through statistical analysis the weighted average "N" greater than 50 blows per foot, in which Site Class C could be used. Other option to consider in the final geotechnical evaluation is obtaining actual shear wave velocity measurements.

5.0 CONSTRUCTION AND DESIGN CONSIDERATIONS

5.1 Excavation and Groundwater Control

Deeper temporary excavations can be supported using soldier piles and lagging. Depending on the depth of excavations and lateral deflection criteria, one or more levels of bracing such as tiebacks may be necessary. If up to three levels of parking is constructed below grade, this will most likely be above the long-term groundwater table, so water infiltration from perched pockets of groundwater into the excavation should be minimal. If a more rigid retaining structure is required to limit movements, and if the site geometric constraints preclude the use of tiebacks, a structural diaphragm wall could be constructed.

Below-grade walls should be designed to resist lateral earth pressures assuming an at-rest condition if they are fixed at the top, or the active lateral earth pressure if allowed to deflect at the top. Based on the assumption that granular soils will be placed as backfill behind these walls, the structures should be designed using the following earth pressure coefficients and unit weights.

Total Unit Weight, γ_t	125 pcf
Angle of Internal Friction, ϕ	30 degrees
At-Rest Earth Pressure Coefficient, K_o	0.50
Active Earth Pressure Coefficient, K_a	0.33
Passive Earth Pressure Coefficient, K_p	3.00

The backfill should consist of clean, free-draining granular material with a maximum of 5% material by weight passing the No. 200 sieve. Surcharge loads from floor slabs and pavement loading should be included in the design. The top of the walls should be braced prior to placing backfill and appropriate compaction equipment sized to minimize the stress on the wall.

All excavations should be carried out in accordance with current federal, state, and local requirements. Temporary slopes in trenches greater than 5 feet in depth should be properly sloped, or supported by means of a trench box or trench shield. A "competent person", as defined by OSHA, should continually evaluate soil conditions during trench excavation and decide on the appropriate trench protection.

5.2 *Construction Monitoring*

The foundation installations must be monitored and evaluated by qualified geotechnical personnel to ensure that the foundations constructed are consistent with the design bearing intended by the geotechnical engineer.

All backfilling should be supervised to ensure that proper materials are employed and that adequate compaction is achieved. Strict quality control guidelines should be followed during the placement of fill materials.

6.0 LIMITATION OF THE INVESTIGATION

This report is intended solely for the Client named. The material in the report reflects our best judgment taking into account the information available to Inspecsol at the time of preparation, and is based on the three (3) boreholes performed under our direction. No portion of this report may be used as a separate entity; it should be read in its entirety. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties.

The preliminary recommendations made in this report are in accordance with our present understanding of the project. We recommend that Inspecsol Engineering Inc. complete a site-specific drilling and testing program, along with detailed geotechnical evaluation and report, once final building configuration, use, type, and column loads are known.

It is also important to emphasize that a soil investigation is, in fact, a random sampling of a site and the comments are based on the results obtained at the borehole locations only. It is, therefore, assumed that these results are representative of the subsoil conditions across the site. Should any conditions at the site be encountered which differ from those found at the borehole locations, we request that we be notified immediately in order to permit a reassessment of our recommendations.

Respectfully Submitted,

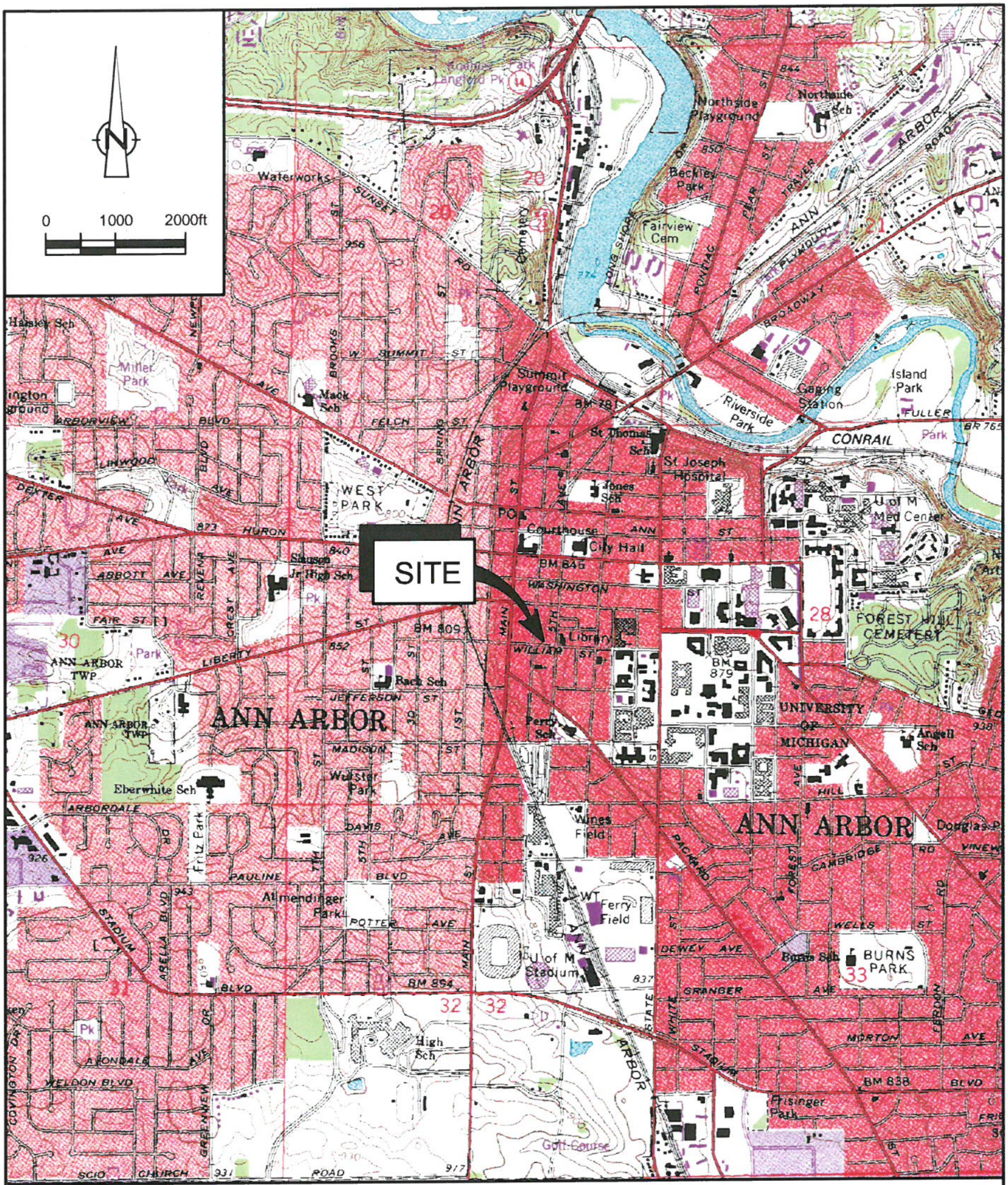
INSPECSOL ENGINEERING INC.

Rebecca E. Bentley

Rebecca E. Bentley, E.I.T.
Staff Engineer

Michael C. Gentner, P.E.
Vice President

FIGURES



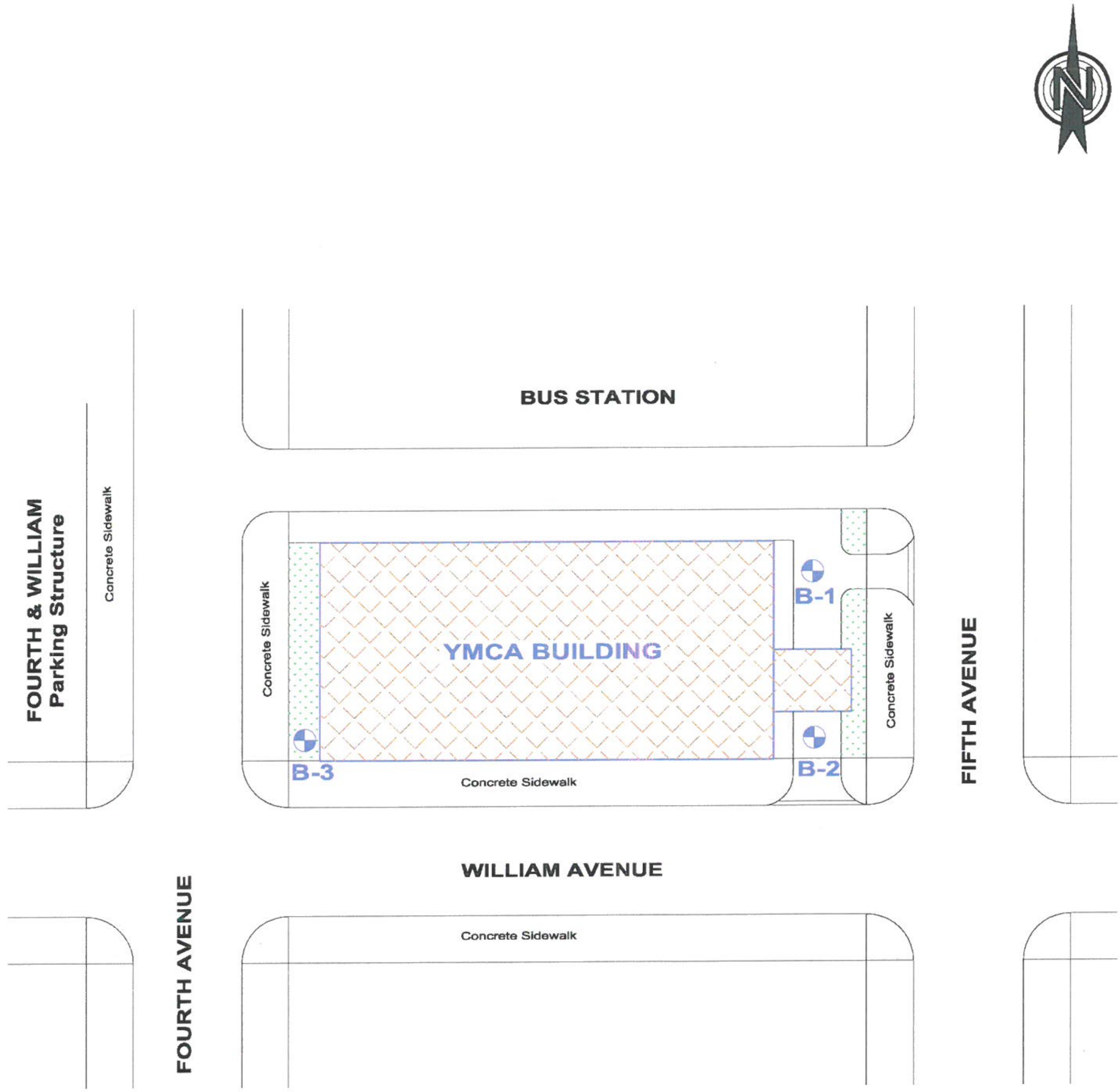
SOURCE: USGS QUADRANGLE MAP;
ANN ARBOR EAST & WEST, MICHIGAN
1965 PHOTO REVISED 1983

figure 1

ANN ARBOR

SITE LOCATION DIAGRAM
Ann Arbor, Michigan

D020036-00(001)GN-DE001 FEB 04/2008



LEGEND:

Approx. Soil Borings B-1 thru B-3 drilled from 12-17-07 to 12-21-07

N.T.S.

HAE
42030 KOPPERNICK ROAD. SUITE 318
CANTON, MICHIGAN 48187
(734) 455-9771/ Fax (734) 455-9774

SOIL BORING LOCATION PLAN

YMCA Building
350 S. Fifth Ave., Ann Arbor, Michigan

Project No.: H-08-803-G	PLATE No.: 1

APPENDIX

1. APPENDIX A: BOREHOLES LOGS
2. APPENDIX B: LABORATORY DATA
3. PRESSUREMETER TEST RESULTS
4. SITE PICTURES

1

APPENDIX A: BOREHOLES LOGS

REFERENCE No.: D020036

ENCLOSURE No.:

INSPECSOL ENGINEERING INC.

BOREHOLE No.: B-1

ELEVATION: 99.6 ft

BOREHOLE REPORT

CLIENT: Haengel & Associates Engineering, Inc.

PROJECT: Former YMCA Development Project

LOCATION: Ann Arbor, MI

DESCRIBED BY: R. Bentley CHECKED BY: M. Gentner

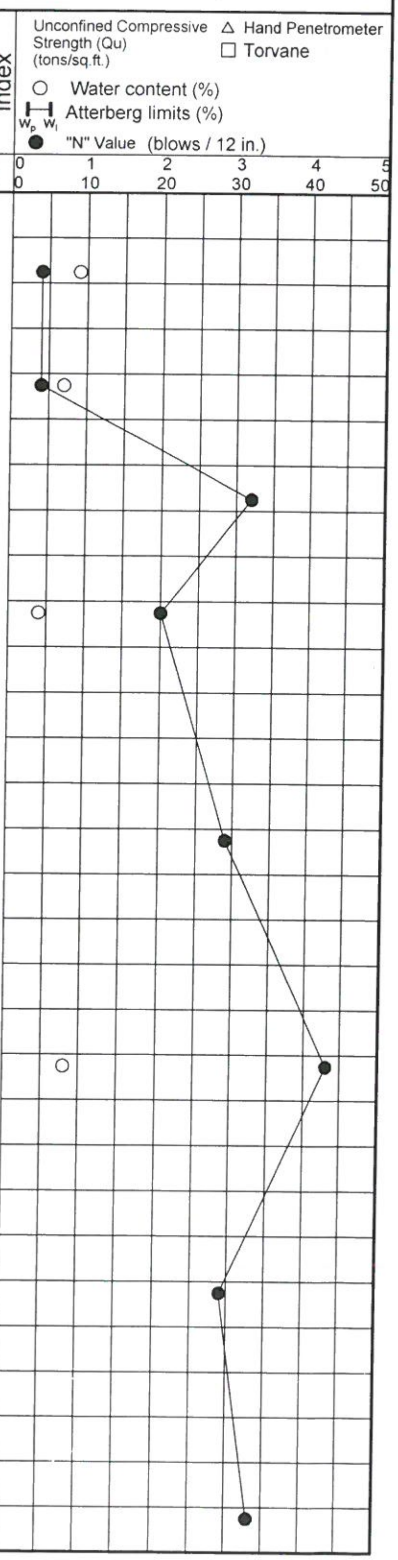
DATE (START): December 17, 2007 DATE (FINISH): December 17, 2007

DRILLER: American Drilling DRILL RIG: CME-75

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- RC - ROCK CORE
- - WATER LEVEL
- GS - GRAB SAMPLE

Depth (Feet)	Elevation (ft)	DESCRIPTION OF SOIL AND BEDROCK	Type and Number	Recovery (%)	Moisture Content	Blows per 6 in.	Penetraion Index (N)
	99.60	GROUND SURFACE					
1.0	98.60	Driller augered to 1.0 ft. before sampling.					
3.0	96.60	(FILL) Fine SAND FILL, trace silt and gravel, very loose, brown, moist.	S-1	56	9	3-2-2	4
5.5	94.10	(SP-SM) Gravelly fine to coarse SAND, trace silt, very loose, brown, moist.	S-2	67	7	2-2-2	4
8.0	91.60	(SP) Fine to coarse SAND, trace gravel, dense, brown, moist.	S-3	67	--	5-11-21	32
		(GP-GM) Fine to coarse GRAVEL, trace silt, compact to dense, brown, moist. Cobbles/boulders noted between 18 and 20 ft.	S-4	67	4	6-9-11	20
			S-5	44	--	11-14-15	29
22.5	77.10		S-6	56	8	31-23-20	43
27.0	72.60	(SP) Fine to coarse SAND, trace gravel, compact, brown, moist.	S-7	56	--	9-13-16	29
30.0		(SP) Fine to medium SAND, trace gravel, dense, brown, moist.	S-8	56	--	27-17-16	33



SOIL LOG WITH GRAPH D020036 YMCA.GPJ CRA PLYMOUTH.GDT 2/4/08

REFERENCE No.: D020036

ENCLOSURE No.:

INSPECSOL ENGINEERING INC.
MEMBER OF THE CRA FAMILY OF COMPANIES

BOREHOLE No.: B-1
ELEVATION: 99.6 ft

BOREHOLE REPORT

CLIENT: Haengel & Associates Engineering, Inc.
PROJECT: Former YMCA Development Project
LOCATION: Ann Arbor, MI
DESCRIBED BY: R. Bentley CHECKED BY: M. Gentner MEG
DATE (START): December 17, 2007 DATE (FINISH): December 17, 2007
DRILLER: American Drilling DRILL RIG: CME-75

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- RC - ROCK CORE
- ▼ - WATER LEVEL
- GS - GRAB SAMPLE

Depth	Elevation (ft)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery	Moisture Content	Blows per 6 in.	Penetraion Index	Unconfined Compressive Strength (Qu) (tons/sq.ft.) △ Hand Penetrometer □ Torvane; ○ Water content (%); W_p W_l Atterberg limits (%); ● "N" Value (blows / 12 in.)
Feet	99.60		GROUND SURFACE			%			N	0 1 2 3 4 5 / 0 10 20 30 40 50
	69.60		END OF BORING Boulder/cobbles shifted into borehole at 20 ft. Driller had difficulty removing drill rods from sampling at 30 ft., and could not re-drill through obstruction. Boring advanced to 15.0 ft with hollow stem augers. Set 15 ft. of 6" diameter casing. Boring advanced from 15.0 to 30.0 ft. with tricone roller bit and wash rotary methods. Borehole backfilled with bentonite-cement grout. WL: None ft. WL (AB): None ft.							
35										
40										
45										
50										
55										

REFERENCE No.: D020036

ENCLOSURE No.:

INSPECSOL ENGINEERING INC.
MEMBER OF THE CRA FAMILY OF COMPANIES

BOREHOLE No.: B-2

ELEVATION: 99.2 ft

BOREHOLE REPORT

CLIENT: Haengel & Associates Engineering, Inc.

PROJECT: Former YMCA Development Project

LOCATION: Ann Arbor, MI

DESCRIBED BY: R. Bentley

CHECKED BY: M. Gentner MG

DATE (START): December 18, 2007

DATE (FINISH): December 19, 2007

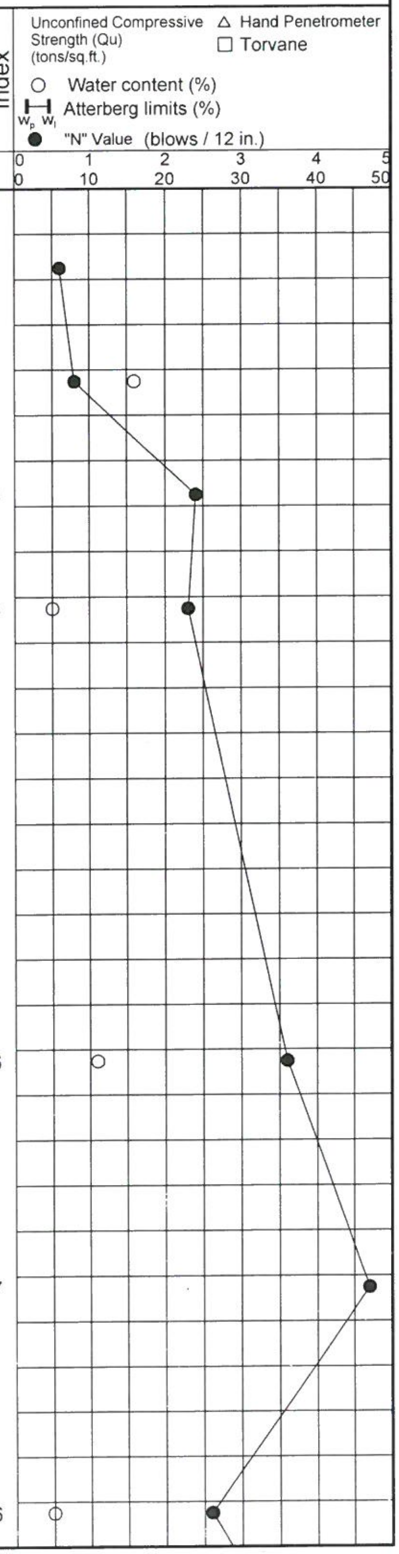
DRILLER: American Drilling

DRILL RIG: CME-75

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- RC - ROCK CORE
- WATER LEVEL
- GS - GRAB SAMPLE

Depth (Feet)	Elevation (ft)	Description of Soil and Bedrock	Type and Number	Recovery (%)	Moisture Content	Blows per 6 in.	Penetration Index (N)
	99.19	GROUND SURFACE					
		Driller augered to 1.0 ft. before sampling.					
1.0	98.19	(FILL) Fine to medium sand FILL with clay, trace gravel, loose, brown, moist.	S-1	56	--	5-3-3	6
3.0	96.19	(FILL) Clayey sand FILL with silt, trace gravel, loose, brown, moist.	S-2	56	16	2-4-4	8
5.5	93.69	(SP) Fine to coarse SAND, trace gravel, compact, brown, moist.	S-3	39	--	6-9-15	24
8.0	91.19	(SP) Fine to medium SAND, trace gravel, compact, brown, moist. Nat. density = 118.6 pcf	S-4*	67	5	8-11-12	23
12.0	87.19	(SP-SM) Gravelly fine to coarse SAND with silt, dense to very dense, brown, moist. Lost casing seal at 15 ft.	S-5	0	--	--	--
			S-6*	67	11	18-15-21	36
			S-7	67	--	11-16-31	47
			S-8*	67	5	9-11-15	26



REFERENCE No.: D020036

ENCLOSURE No.:

INSPECSOL ENGINEERING INC.
MEMBER OF THE CRA FAMILY OF COMPANIES

BOREHOLE No.: B-2

ELEVATION: 99.2 ft

BOREHOLE REPORT

CLIENT: Haengel & Associates Engineering, Inc.

PROJECT: Former YMCA Development Project

LOCATION: Ann Arbor, MI

DESCRIBED BY: R. Bentley

CHECKED BY: M. Gentner MCM

DATE (START): December 18, 2007

DATE (FINISH): December 19, 2007

DRILLER: American Drilling

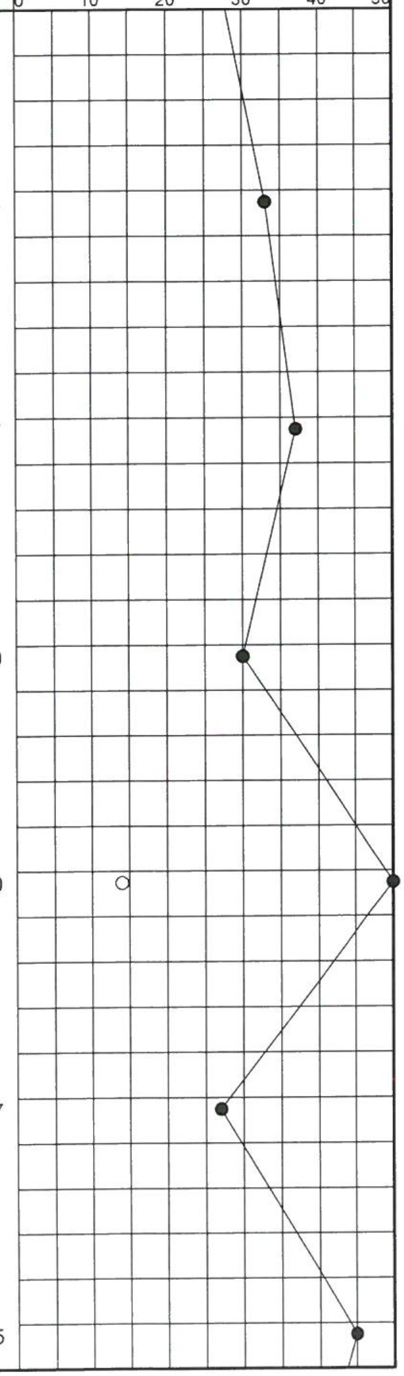
DRILL RIG: CME-75

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- RC - ROCK CORE
- WATER LEVEL
- GS - GRAB SAMPLE

Depth	Elevation (ft)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery	Moisture Content	Blows per 6 in.	Penetraion Index
Feet	99.19		GROUND SURFACE			%			N
	37.19		62.0 (SP) Fine to coarse SAND, trace gravel, dense, brown, moist.						
				SS	S-15	67	--	11-17-16	33
65									
				SS	S-16	56	--	14-16-21	37
70									
				SS	S-17	67	--	11-14-16	30
75									
	22.19		77.0 (SP-SM) Fien to medium SAND, trace gravel and silt, very dense, brown, moist.						
				SS	S-18	67	14	18-21-29	50
80									
	17.19		82.0 (SP) Fine to coarse SAND, trace gravel, dense, brown, moist.						
				SS	S-19	67	--	20-11-16	27
85									
				SS	S-20	67	--	19-22-23	45

Unconfined Compressive Strength (Qu) (tons/sq.ft.) △ Hand Penetrometer □ Torvane
○ Water content (%)
W_p W_l Atterberg limits (%)
● "N" Value (blows / 12 in.)



REFERENCE No.: D020036 ENCLOSURE No.:

INSPECSOL ENGINEERING INC.
MEMBER OF THE CRA FAMILY OF COMPANIES

BOREHOLE No.: B-2
ELEVATION: 99.2 ft

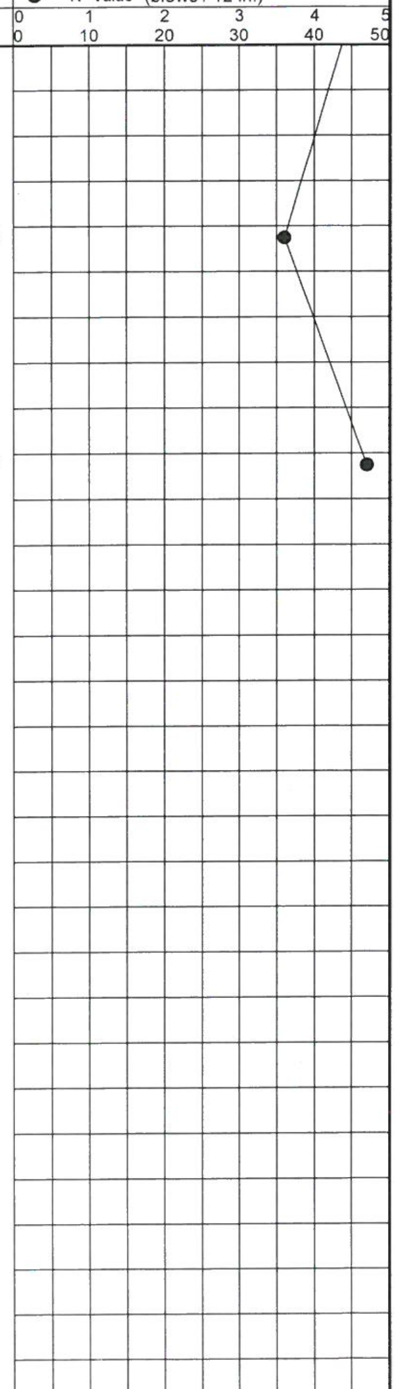
BOREHOLE REPORT

CLIENT: Haengel & Associates Engineering, Inc.
PROJECT: Former YMCA Development Project
LOCATION: Ann Arbor, MI
DESCRIBED BY: R. Bentley CHECKED BY: M. Gentner
DATE (START): December 18, 2007 DATE (FINISH): December 19, 2007
DRILLER: American Drilling DRILL RIG: CME-75

LEGEND
- SS - SPLIT SPOON
- ST - SHELBY TUBE
- RC - ROCK CORE
- ▼ - WATER LEVEL
- GS - GRAB SAMPLE

Depth (Feet)	Elevation (ft)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery (%)	Moisture Content	Blows per 6 in.	Penetraion Index (N)
	99.19		GROUND SURFACE						
					S-21	67	--	20-17-19	36
95									
					S-22	67	--	21-23-24	47
100	-0.81		100.0						
			END OF BORING Boring advanced to 13.5 ft with hollow stem augers. Set 17 ft. of 6" diameter casing. Boring advanced from 17 to 100 ft. with tricone roller bit and wash rotary methods. * Indicates 3" brass liner insert in split spoon. Borehole backfilled with bentonite-cement grout. WL: None ft. WL (AB): None ft.						
105									
110									
115									

Unconfined Compressive Strength (Qu) (tons/sq.ft.) △ Hand Penetrometer □ Torvane
○ Water content (%)
W_p W_l Atterberg limits (%)
● "N" Value (blows / 12 in.)



SOIL LOG WITH GRAPH D020036 YMCA.GPJ CRA PLYMOUTH.GDT 2/4/08

REFERENCE No.: D020036 ENCLOSURE No.:

INSPECSOL ENGINEERING INC. MEMBER OF THE CRA FAMILY OF COMPANIES

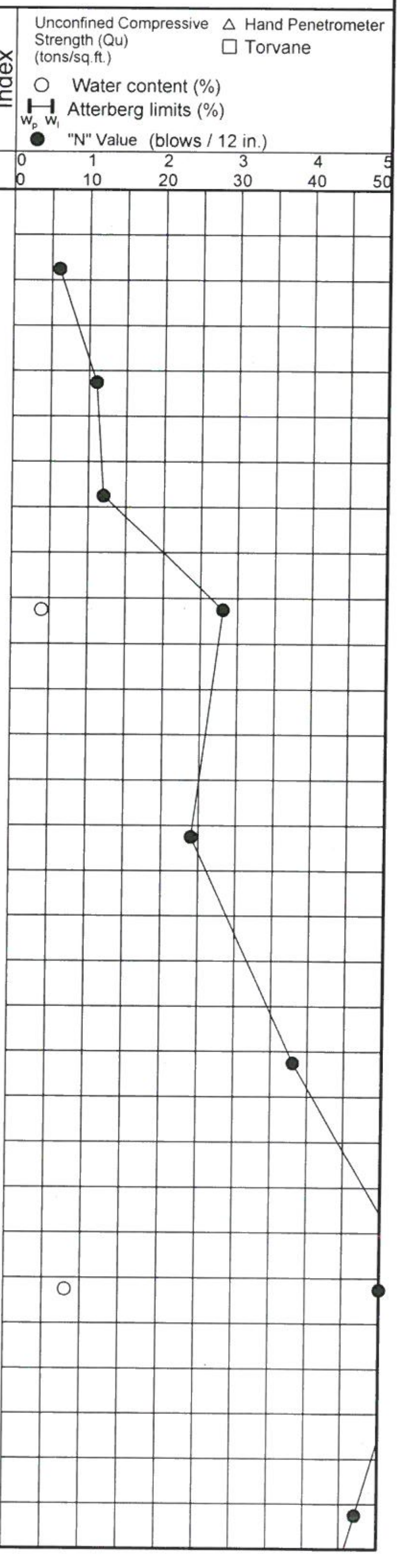
BOREHOLE No.: B-3
ELEVATION: 94.0 ft

BOREHOLE REPORT

CLIENT: Haengel & Associates Engineering, Inc.
PROJECT: Former YMCA Development Project
LOCATION: Ann Arbor, MI
DESCRIBED BY: R. Bentley CHECKED BY: M. Gentner MG
DATE (START): December 20, 2007 DATE (FINISH): December 21, 2007
DRILLER: American Drilling DRILL RIG: CME-75

LEGEND
- SS - SPLIT SPOON
- ST - SHELBY TUBE
- RC - ROCK CORE
- ▼ - WATER LEVEL
- GS - GRAB SAMPLE

Depth (Feet)	Elevation (ft)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery (%)	Moisture Content	Blows per 6 in.	Penetraion Index (N)
	94.03		GROUND SURFACE						
			Driller augered to 1.0 ft. before sampling.						
1.0	93.03		(SP) Fine to medium SAND, trace gravel, loose to compact, brown, moist.						
				SS	S-1	67	--	3-3-3	6
				SS	S-2	100	--	3-5-6	11
5.0	89.03		(SP) Fine to coarse SAND, trace gravel, compact to dense, brown, moist.						
				SS	S-3	56	--	4-4-8	12
				SS	S-4*	67	4	14-14-14	28
				SS	S-5	67	--	6-10-14	24
				SS	S-6	6	--	11-17-21	38
22.0	72.03		(GM) Silty GRAVEL with fine to coarse sand, very dense, brown, moist.						
				SS	S-7*	67	8	15-31-24	56
27.0	67.03		(SP) Fine to coarse SAND, trace gravel, dense, brown, moist.						
				SS	S-8	100	--	12-19-28	47



REFERENCE No.: D020036 ENCLOSURE No.:

INSPECSOL ENGINEERING INC.

BOREHOLE No.: B-3
ELEVATION: 94.0 ft

BOREHOLE REPORT

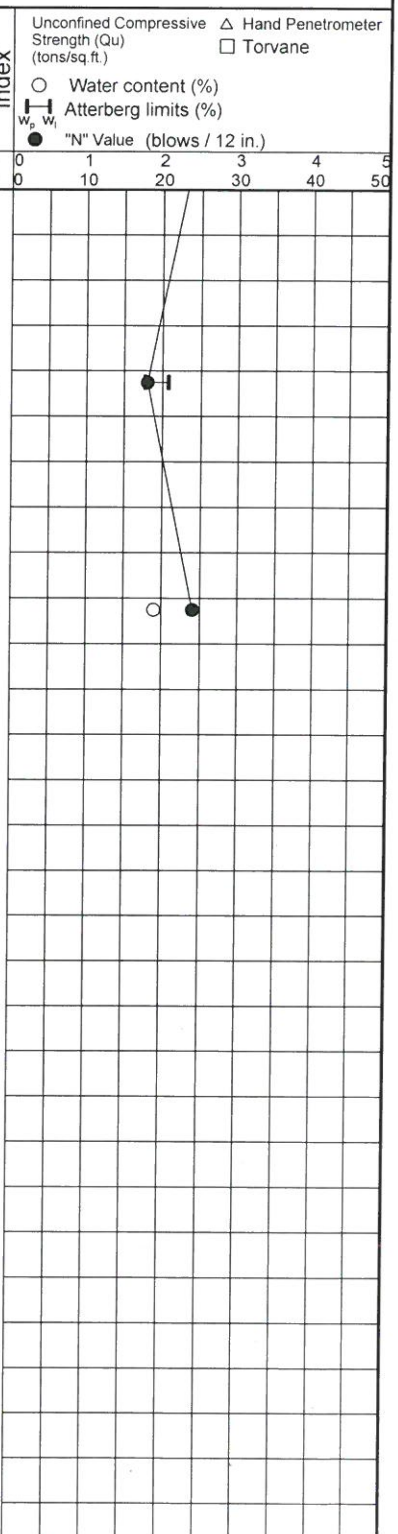
CLIENT: Haengel & Associates Engineering, Inc.
PROJECT: Former YMCA Development Project
LOCATION: Ann Arbor, MI
DESCRIBED BY: R. Bentley CHECKED BY: M. Gentner [illegible]
DATE (START): December 20, 2007 DATE (FINISH): December 21, 2007

LEGEND

- ⊠ SS - SPLIT SPOON
- ▨ ST - SHELBY TUBE
- ◫ RC - ROCK CORE
- ▼ - WATER LEVEL
- ▩ GS - GRAB SAMPLE

DRILLER: American Drilling DRILL RIG: CME-75

Depth	Elevation (ft)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery	Moisture Content	Blows per 6 in.	Penetraion Index
Feet	94.03		GROUND SURFACE			%			N
				⊠	S-18	100	--	6-7-11	18
95									
				⊠	S-19	100	19	9-11-13	24
100	-5.97		100.0						
			END OF BORING Boring advanced to 13.5 ft with hollow stem augers. Set 22 ft. of 6" diameter casing. Boring advanced from 13.5 to 100 ft. with tricone roller bit and wash rotary methods. 50% mud loss between 30 and 40 ft. * Indicates 3" brass liner insert in split spoon. Borehole backfilled with bentonite-cement grout. WL: None ft. WL (AB): None ft.						
105									
110									
115									



SOIL CLASSIFICATION SYSTEM (MODIFIED U.S.C.S.)

MAJOR DIVISIONS				GROUP SYMBOL	TYPICAL DESCRIPTION
HIGHLY ORGANIC SOILS				PT	PEAT AND OTHER HIGHLY ORGANIC SOILS
COARSE-GRAINED SOILS (MORE THAN HALF BY WEIGHT LARGER THAN THE NO. 200 SIEVE SIZE)	GRAVELS	MORE THAN HALF OF COARSE FRACTION LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS	GW	WELL GRADED GRAVEL, GRAVEL-SAND MIXTURES, < 5% FINES
				GP	POORLY GRADED GRAVELS AND GRAVEL-SAND MIXTURES, < 5% FINES
			DIRTY GRAVELS	GM	SILTY GRAVELS, GRAVE-SAND-SILT MIXTURES, > 12% FINES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES, >12% FINES
	SANDS	MORE THAN HALF OF COARSE FRACTION SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS	SW	WELL GRADED SANDS, GRAVELLY SANDS, < 5% FINES
				SP	POORLY GRADED SANDS, OR GRAVELLY SAND, < 5% FINES
			DIRTY SANDS	SM	SILTY SANDS, SAND-SILT MIXTURES, > 12% FINES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES, > 12% FINES
FINE -GRAINED SOILS (MORE THAN HALF BY WEIGHT PASSES NO 200 SIEVE SIZE)	SILTS BELOW "A" LINE ON PLASTICITY CHART; NEGLIGIBLE ORGANIC CONTENT			ML	INORGANIC SILTS AND VERY FINE SAND, ROCK FLOUR, SILTY SANDS OF SLIGHT PLACTICITY
				MH	INORGANIC SILTY, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS
	CLAYS ABOVE "A" LINE ON PLASTICITY CHART; NEGLIGIBLE ORGANIC CONTENT			CL	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY OR SILTY CLAYS, LEAN CLAYS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
	ORGANIC SILTS & ORGANIC CLAYS BELOW "A" LINE ON PLASTICITY CHART			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
				OH	ORGANIC CLAYS OF HIGH PLASTICITY

INSPECSOL ENGINEERING INC.
MEMBER OF THE CRA FAMILY OF COMPANIES

CONVENTIONAL SOIL DESCRIPTIONS

NON-COHESIVE (GRANULAR) SOIL

RELATIVE DENSITY	BLOWS PER FOOT (N-VALUE)
Very loose	< 5
Loose	5 to 9
Compact	10 to 29
Dense	30 to 50
Very Dense	> 50

COHESIVE (CLAYEY) SOIL

CONSISTENCY	UNCONFINED COMPRESSIVE STRENGTH,Qu (tsf)
Very Soft	< 0.25
Soft	0.25 to 0.49
Firm	0.50 to 0.99
Stiff	1.00 to 1.99
Very Stiff	2.00 to 3.99
Hard	> 4.00

GRAIN SIZE CLASSIFICATION

COBBLES	Greater than 3 inches (76 mm)
GRAVEL	3 in. to No. 4 (4.76 mm)
Coarse Gravel	3 in. to 3/4 in.
Fine Gravel	3/4 in to No. 4 (4.76 mm)
SAND	No. 4 (4.76 mm) to No. 200 (.0.074 mm)
Coarse Sand	No. 4 (4.76 to No. 10 (2.0 mm)
Medium Sand	No. 10 (2.0 mm) to No. 40 (0.42 mm)
Fine Sand	No. 40 (0.42 mm) to No. 200 (0.074 mm)
SILT	No. 200 (0.074 mm) to 0.002 mm
CLAY	Less than 0.002 mm

NOTE: The "No. __" refers to the standard sieve sizes.

COMPONENT PERCENTAGE DESCRIPTORS

Noun(s) (e.g. SAND and GRAVEL)	35 to 50%
Adjective (e.g. SANDY)	20 to 35%
With	10 to 20%
Trace	Less than 10%

Stratified	Blocky
Laminated	Lenses/Seams
Fissured	Homogeneous

2

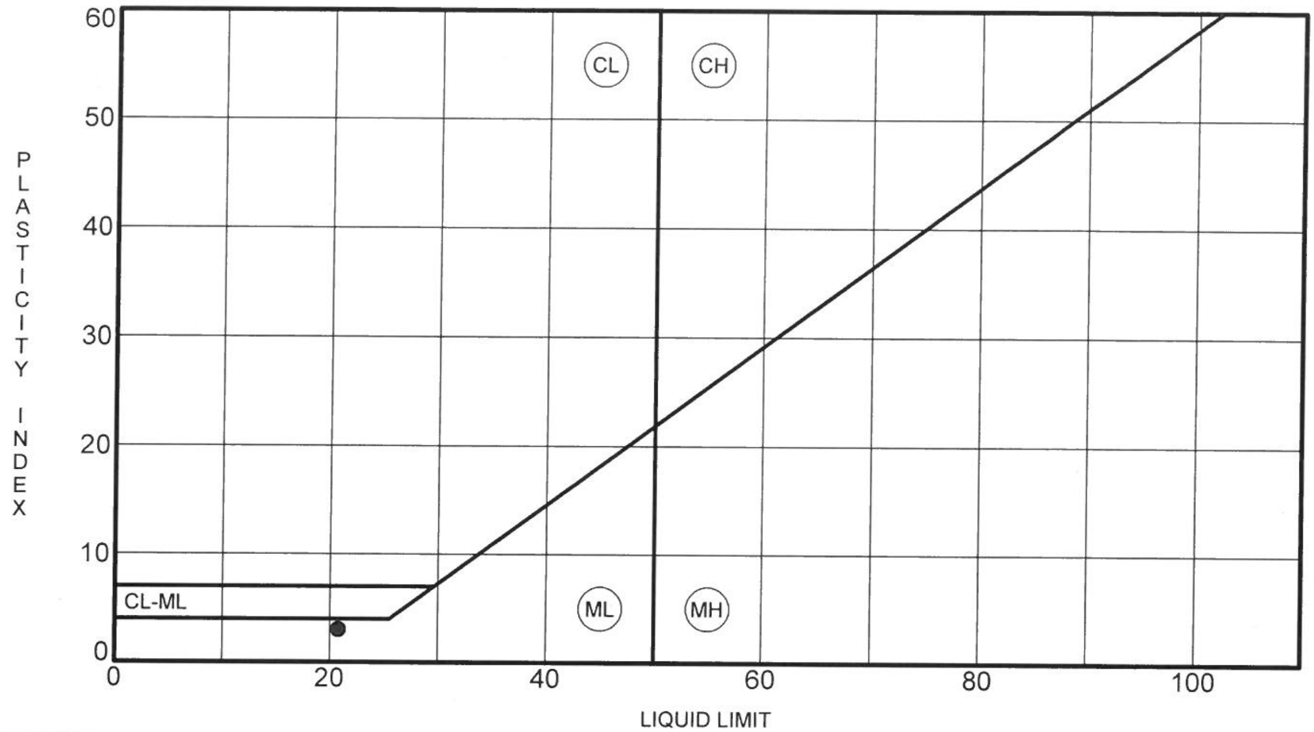
APPENDIX B: LABORATORY DATA

Borehole	Depth ()	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Size (mm)	%<#200 Sieve	Class-ification	Water Content (%)	Dry Density (pcf)	Satur-ation (%)	Void Ratio
B-1	1							9.0			
B-1	3.5							6.6			
B-1	6.5				37.5	10		4.2			
B-1	8.5							4.3			
B-1	18.5				37.5	8		7.8			
B-2	3.5							15.5			
B-2	8.5							4.8	118.6		
B-2	18.5							10.5			
B-2	28.5				37.5	10		4.7	117.4		
B-2	38.5							9.2	148.0		
B-2	48.5							14.2	114.4		
B-2	78.5				12.5	6		13.6			
B-3	1.5							3.7			
B-3	8.5							4.0			
B-3	23.5				37.5	32		7.9			
B-3	38.5							14.9			
B-3	58.5							11.4			
B-3	68.5							17.2			
B-3	78.5							16.4	125.3		
B-3	88.5				0.15	95		24.6			
B-3	93.5	21	18	3							
B-3	98.5							19.4			

INSPECSOL ENGINEERING INC.
MEMBER OF THE CRA FAMILY OF COMPANIES

Summary of Laboratory Results

Project Name: Former YMCA Development Project
Project Number: D020036
Client: Haengel & Associates Engineering, Inc.
Location: Ann Arbor, MI



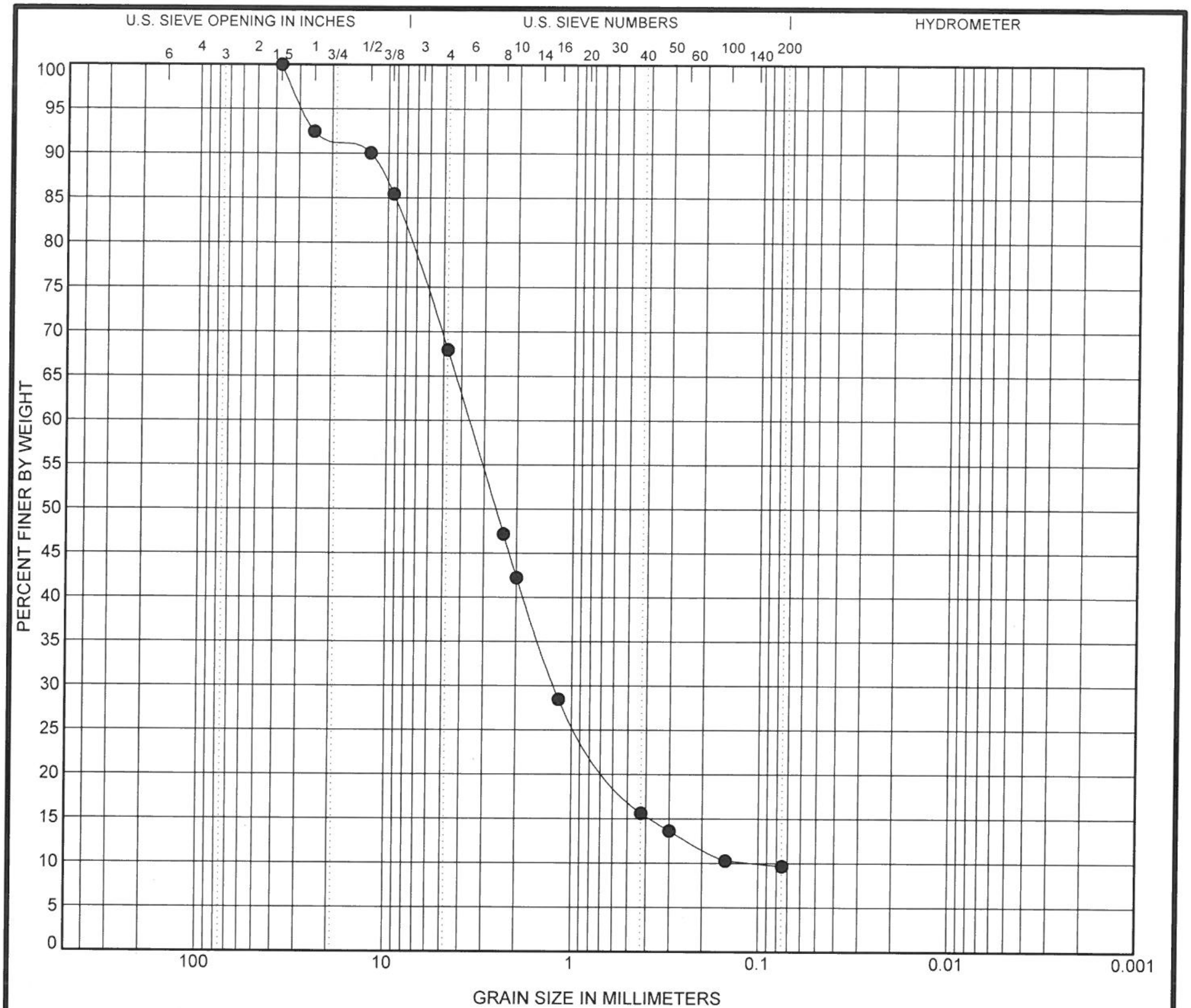
	Specimen Identification		LL	PL	PI	Fines	Classification
●	**B-3**	**93.5**	**21**	**18**	**3**		**(ML) Clayey SILT, compact, gray, moist.**

ATTERBERG LIMITS D020036 YMCA.GPJ CRA PLYMOUTH.GDT 2/4/08

INSPECSOL ENGINEERING INC.
MEMBER OF THE CRA FAMILY OF COMPANIES

ATTERBERG LIMITS RESULTS

Project Name: Former YMCA Development Project
Project Number: D020036
Client: Haengel & Associates Engineering, Inc.
Location: Ann Arbor, MI



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification		Classification	LL	PL	PI	Cc	Cu
● B-1	6.5	(SP-SM) Gravelly fine to coarse SAND, trace silt, brown.				4.00	33.75

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● B-1	6.5	37.5	3.633	1.251	0.108	32.0	58.3	9.7	

INSPECSOL ENGINEERING INC.
MEMBER OF THE CRA FAMILY OF COMPANIES

GRAIN SIZE DISTRIBUTION

Project Name: Former YMCA Development Project
Project Number: D020036
Client: Haengel & Associates Engineering, Inc.
Location: Ann Arbor, MI

GRAIN SIZE D020036 YMCA.GPJ CRA PLYMOUTH.GDT 2/4/08

Test by: DE

Checked by: [signature]

INSPECSOL ENGINEERING INC.
MEMBER OF THE CRA FAMILY OF COMPANIES

PARTICLE-SIZE ANALYSIS OF SOILS (ASTM D 422-63)

CLIENT: Haengel & Associates Engineering, Inc. LAB NO: 115
PROJECT / SITE: YMCA Development Project PROJECT NO: D020036

BORING NO: B1 SAMPLE NO: S-3 DEPTH: 6' - 7.5'
SOIL DESCRIPTION: (SP-SM) Gravelly fine to coarse SAND, trace silt, brown.

APPARATUS:
Balance No: ______ Sieve No: ______ Hydrometer No: ______
Oven No: ______ 1000ml Cylinder No.: ______ Hydrometer Jar No: ______

SIEVING

TOTAL DRY WEIGHT: 306.9 g
Weight after washing: 283.6 g
Loss by wash: 23.3 g

DRY ______
HUMID ______
% PASSING #10 ______

	SIEVE SIZE	WEIGHT RETAINED (g)	% RETAINED	% TOTAL PASSING
GRAVEL	3"	0.0	0.0%	100.0%
	1"	23.1	7.5%	92.5%
	3/4"	0.0	0.0%	92.5%
	1/2"	5.7	1.9%	90.6%
	3/8"	14.1	4.6%	86.0%
	#4	53.7	17.5%	68.5%
SAND	#8	63.8	20.8%	47.7%
	#10	15.2	5.0%	42.8%
	#16	42.2	13.8%	29.0%
	#40	39.4	12.8%	16.2%
	#50	6.1	2.0%	14.2%
	#100	10.3	3.4%	10.9%
	#200	8.1	2.6%	8.2%
	pan	1.9	8.2%	0.0%

HYDROMETER

°C	T min	R	δR	R - δR	L	L / T	K	D	P%
	0.5								
	1								
	2								
	5								
	15								
	30								
	60								
	120								
	240								
	480								
	1440								

WATER CONTENT		
CAN No:	31B	
Soil wet + Tare	498.0	
Soil dry + Tare	485.1	
Weight water	12.9	
Tare	178.2	
Weight dry soil	306.9	
W%	4.2	

$$P\% = \frac{(R - \delta R)\,a}{Ms} \times 100$$

Dmm = K √L/T L is function of (R + meniscus) R'

Dr = ______ ☐ measured ☐ estimated

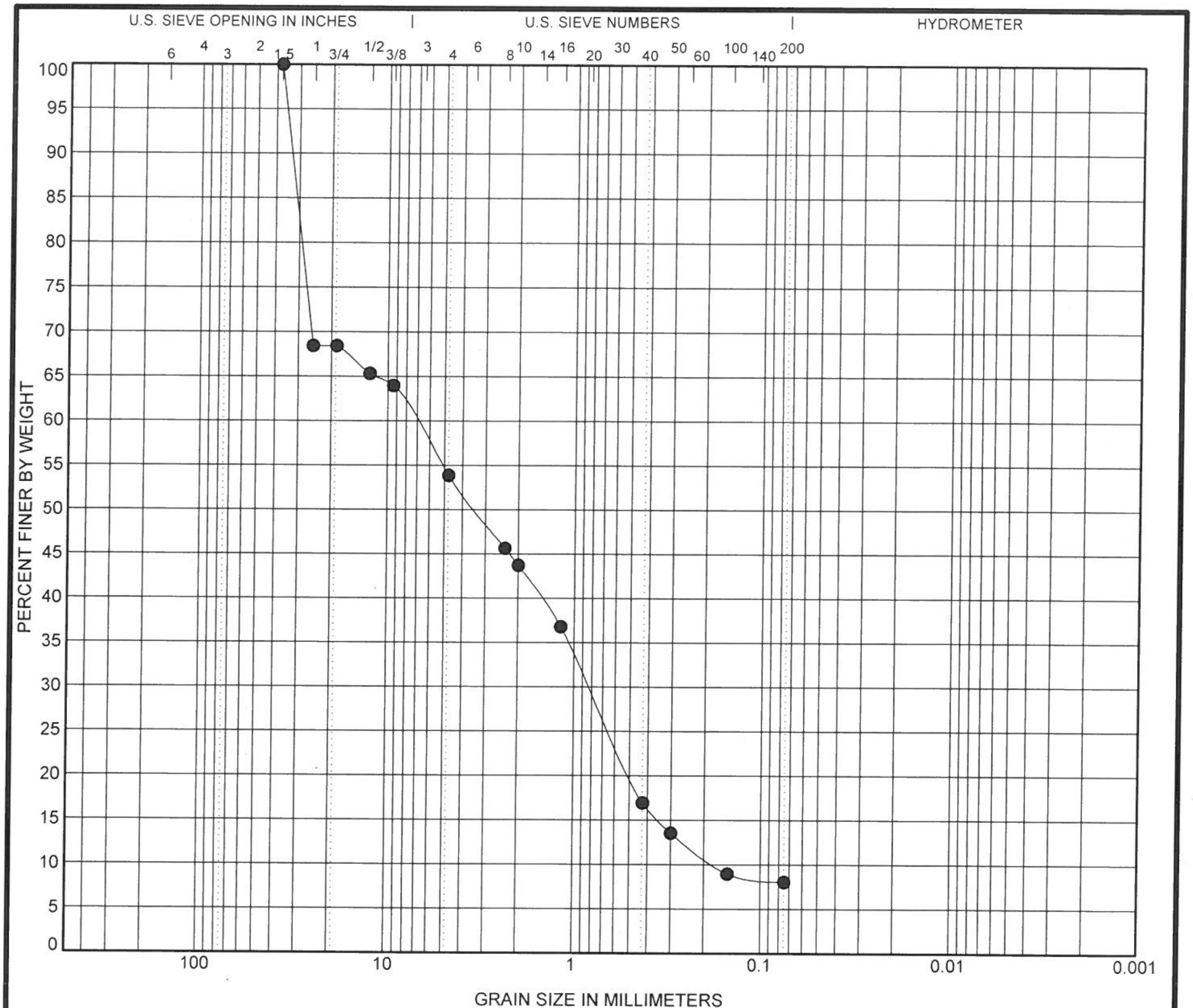
a = ______

DISPERSING AGENT: ______

CONCENTRATION: ______ g/L

REMARKS

PERFORMED BY: Dan Kribs DATE: 1/10/2008
CHECKED BY: [signature] DATE: 1/10/08



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification		Classification	LL	PL	PI	Cc	Cu
● B-1	18.5	(GP-GM) Fine to coarse GRAVEL, trace silt, brown.				0.55	40.94

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● B-1	18.5	37.5	7.167	0.832	0.175	46.1	45.8	8.1	

GRAIN SIZE D020036 YMCA.GPJ CRA PLYMOUTH.GDT 2/4/08

INSPECSOL ENGINEERING INC.
MEMBER OF THE CRA FAMILY OF COMPANIES

GRAIN SIZE DISTRIBUTION

Project Name: Former YMCA Development Project
Project Number: D020036
Client: Haengel & Associates Engineering, Inc.
Location: Ann Arbor, MI

Test by: DK **Checked by:** [illegible]

iNSPECSOL ENGINEERING INC.
MEMBER OF THE CRA FAMILY OF COMPANIES

PARTICLE-SIZE ANALYSIS OF SOILS (ASTM D 422-63)

CLIENT:	Haengel & Associates Engineering, Inc.	LAB NO:	115
PROJECT / SITE:	YMCA Development Project	PROJECT NO:	D020036

BORING NO: B1 SAMPLE NO: S-6 DEPTH: 18.5' - 20'

SOIL DESCRIPTION: (GP-GM) Fine to coarse GRAVEL, trace silt, brown.

APPARATUS:

Balance No: ____ Sieve No: ____ Hydrometer No: ____

Oven No: ____ 1000ml Cylinder No.: ____ Hydrometer Jar No: ____

SIEVING

TOTAL DRY WEIGHT: 238.3 g

Weight after washing: 225.2 g

Loss by wash: 13.1 g

DRY ____

HUMID ____

% PASSING #10 ____

	SIEVE SIZE	WEIGHT RETAINED (g)	% RETAINED	% TOTAL PASSING
GRAVEL	3"	0.0	0.0%	100.0%
	1"	75.2	31.6%	68.4%
	3/4"	0.0	0.0%	68.4%
	1/2"	7.4	3.1%	65.3%
	3/8"	3.2	1.3%	64.0%
	#4	24.1	10.1%	53.9%
SAND	#8	19.6	8.2%	45.7%
	#10	4.6	1.9%	43.7%
	#16	16.5	6.9%	36.8%
	#40	47.3	19.8%	17.0%
	#50	8.1	3.4%	13.6%
	#100	10.9	4.6%	9.0%
	#200	6.1	2.6%	6.4%
	pan	2.2	6.4%	0.0%

HYDROMETER

˚C	T min	R	δR	R - δR	L	L / T	K	D	P%
	0.5								
	1								
	2								
	5								
	15								
	30								
	60								
	120								
	240								
	480								
	1440								

WATER CONTENT

CAN No:	PT-3	
Soil wet + Tare	275.2	
Soil dry + Tare	254.7	
Weight water	20.5	
Tare	16.4	
Weight dry soil	238.3	
W%	8.6	

$$P\% = \frac{(R - \delta R)\ a}{Ms} \times 100$$

$Dmm = K\sqrt{L/T}$ L is function of (R + meniscus) R'

Dr = ____ ☐ measured ☐ estimated

a = ____

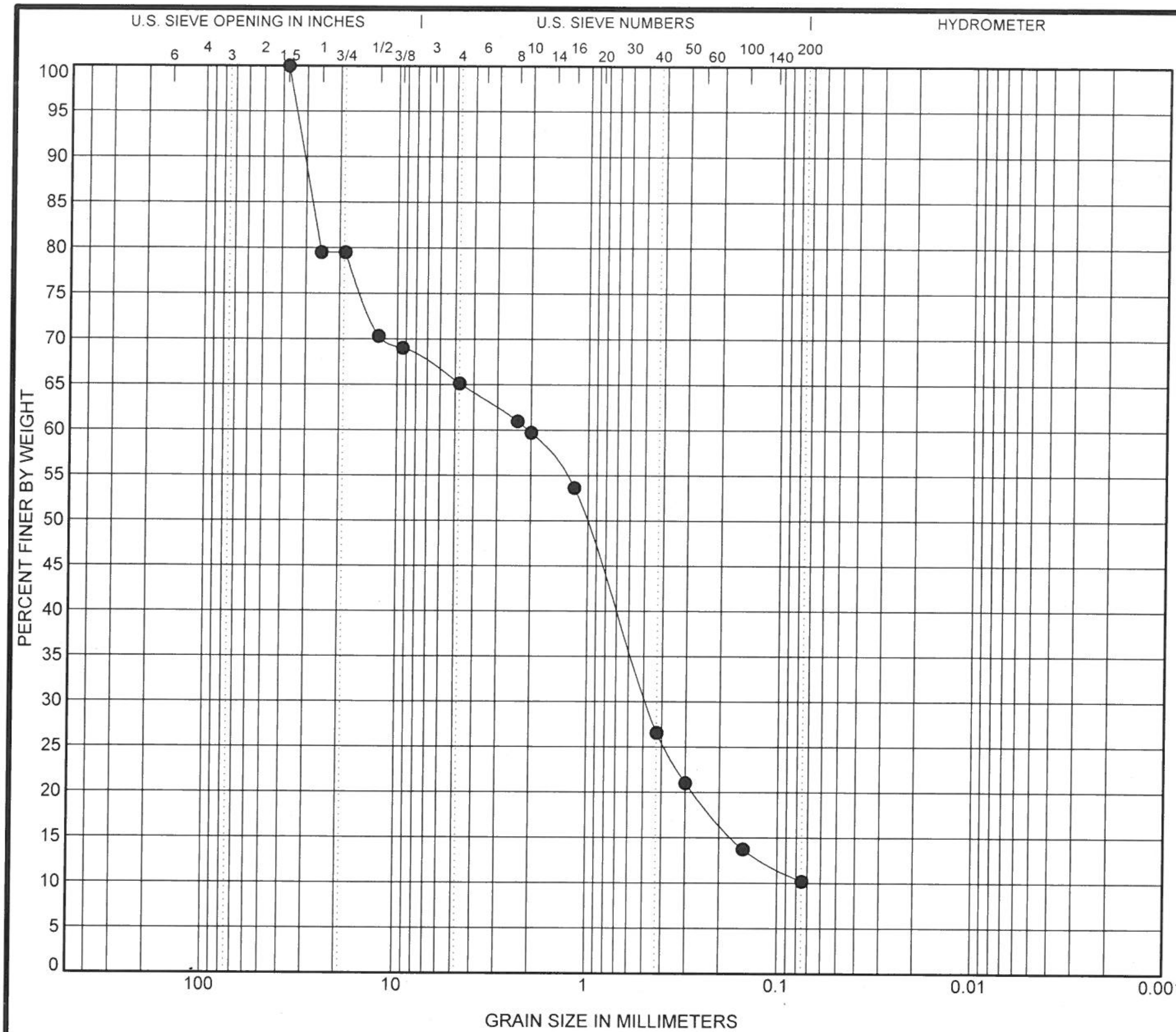
DISPERSING AGENT: ____

CONCENTRATION: ____ g/L

REMARKS: Possible cobble in sample - results may not be representative

PERFORMED BY: Dan Kribs DATE: 1/10/2008

CHECKED BY: [signature] DATE: 1/10/08



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification		Classification	LL	PL	PI	Cc	Cu
● B-2	28.5	(SP-SM) Gravelly fine to coarse SAND with silt, brown.				1.57	28.74

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● B-2	28.5	37.5	2.072	0.484		34.9	54.9	10.2	

GRAIN SIZE D020036 YMCA.GPJ CRA PLYMOUTH.GDT 2/4/08

INSPECSOL ENGINEERING INC.
MEMBER OF THE CRA FAMILY OF COMPANIES

GRAIN SIZE DISTRIBUTION

Project Name: Former YMCA Development Project
Project Number: D020036
Client: Haengel & Associates Engineering, Inc.
Location: Ann Arbor, MI

Test by: DK Checked by: [illegible signature]

INSPECSOL ENGINEERING INC.
MEMBER OF THE CRA FAMILY OF COMPANIES

PARTICLE-SIZE ANALYSIS OF SOILS (ASTM D 422-63)

CLIENT: Haengel & Associates Engineering, Inc. LAB NO: 115
PROJECT / SITE: YMCA Development Project PROJECT NO: D020036

BORING NO: B-2 SAMPLE NO: S-8 DEPTH: 28.5' - 30'
SOIL DESCRIPTION: (SP-SM) Gravelly fine to coarse SAND with silt, brown.

APPARATUS:
Balance No: ______ Sieve No: ______ Hydrometer No: ______
Oven No: ______ 1000ml Cylinder No.: ______ Hydrometer Jar No: ______

SIEVING

TOTAL DRY WEIGHT: 223.5 g
Weight after washing: 202.7 g
Loss by wash: 20.8 g

DRY ______
HUMID ______
% PASSING #10 ______

	SIEVE SIZE	WEIGHT RETAINED (g)	% RETAINED	% TOTAL PASSING
GRAVEL	3"	0.0	0.0%	100.0%
	1"	45.8	20.5%	79.5%
	3/4"	0.0	0.0%	79.5%
	1/2"	20.5	9.2%	70.3%
	3/8"	2.9	1.3%	69.0%
	#4	8.7	3.9%	65.1%
SAND	#8	9.3	4.2%	61.0%
	#10	2.8	1.3%	59.7%
	#16	13.6	6.1%	53.6%
	#40	60.5	27.1%	26.6%
	#50	12.4	5.5%	21.0%
	#100	16.3	7.3%	13.7%
	#200	7.9	3.5%	10.2%
	pan	2.0	10.2%	0.0%

HYDROMETER

°C	T min	R	δR	R - δR	L	L / T	K	D	P%
	0.5								
	1								
	2								
	5								
	15								
	30								
	60								
	120								
	240								
	480								
	1440								

WATER CONTENT

CAN No:	PT-6	
Soil wet + Tare	256.8	
Soil dry + Tare	240.1	
Weight water	16.7	
Tare	16.6	
Weight dry soil	223.5	
W%	7.5	

$$P\% = \frac{(R - \delta R)\, a}{Ms} \times 100$$

$Dmm = K\sqrt{L/T}$ L is function of (R + meniscus) R'

Dr = ______ ☐ measured ☐ estimated

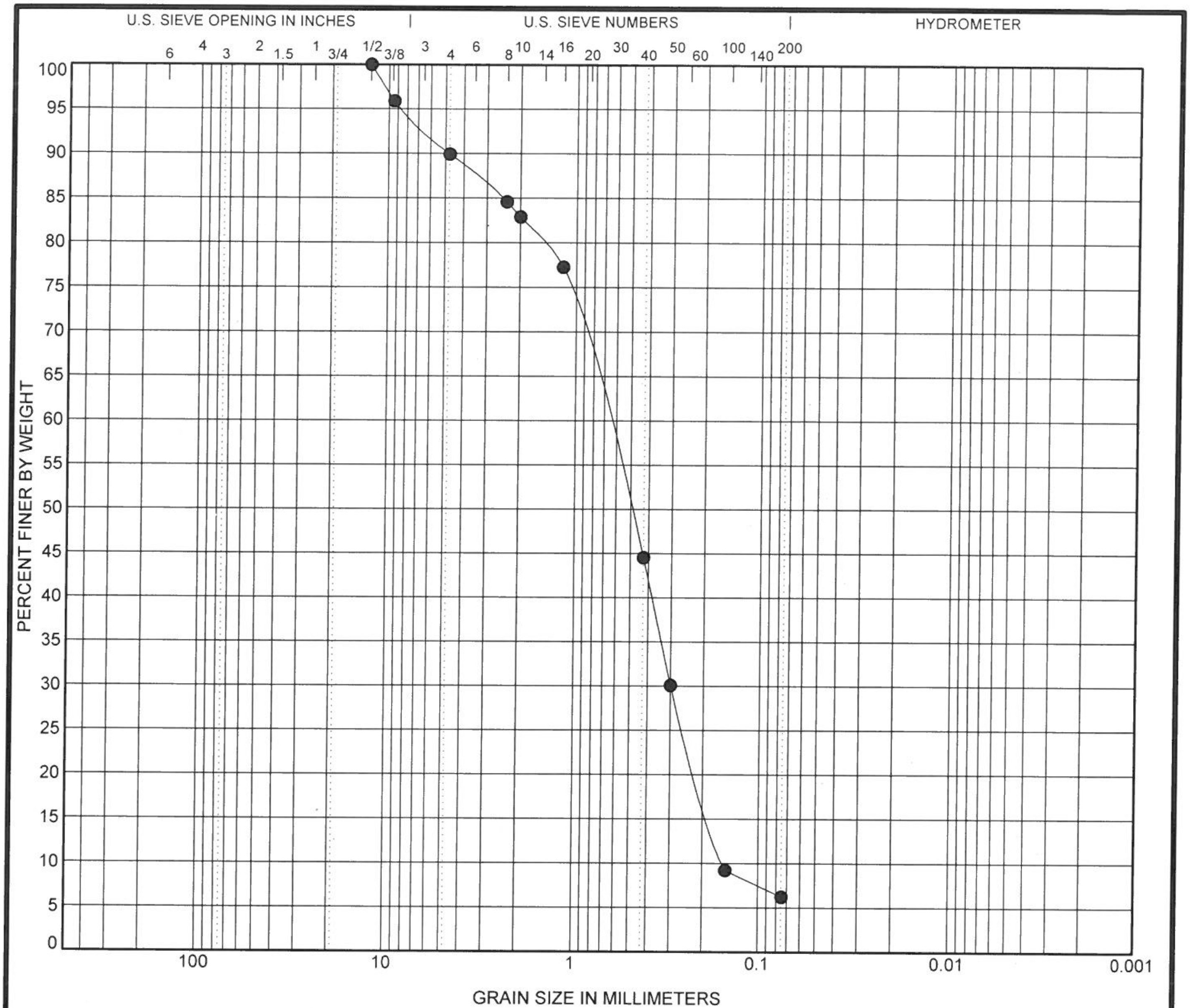
a = ______

DISPERSING AGENT: ______

CONCENTRATION: ______ g/L

REMARKS

PERFORMED BY: Dan Kribs DATE: 1/10/2008
CHECKED BY: [signature] DATE: 1/10/08



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	B-2	78.5	**(SP-SM) Fine to medium SAND with gravel, trace silt, brown.**				**0.84**	**4.47**

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-2	78.5	12.5	0.689	0.299	0.154	10.1	83.7	6.2	

GRAIN SIZE D020036 YMCA.GPJ CRA PLYMOUTH.GDT 2/4/08

INSPECSOL ENGINEERING INC.
MEMBER OF THE CRA FAMILY OF COMPANIES

GRAIN SIZE DISTRIBUTION

Project Name: Former YMCA Development Project
Project Number: D020036
Client: Haengel & Associates Engineering, Inc.
Location: Ann Arbor, MI

Test by: DE Checked by: [illegible]

iNSPECSOL ENGINEERING INC.
MEMBER OF THE CRA FAMILY OF COMPANIES

PARTICLE-SIZE
ANALYSIS OF SOILS
(ASTM D 422-63)

CLIENT:	Hengal & Associates Engineering.	LAB NO:	115
PROJECT / SITE:	YMCA Development Project	PROJECT NO:	D020036

BORING NO: B-2 SAMPLE NO: S-18 DEPTH: 78.5' - 80'

SOIL DESCRIPTION: (SP-SM) Fine to medium SAND with gravel, trace silt, brown.

APPARATUS:

Balance No: ____ Sieve No: ____ Hydrometer No: ____

Oven No: ____ 1000ml Cylinder No.: ____ Hydrometer Jar No: ____

SIEVING

TOTAL DRY WEIGHT: 177.8 g

Weight after washing: 167.8 g

Loss by wash: 10.0 g

DRY ____

HUMID ____

% PASSING #10 ____

	SIEVE SIZE	WEIGHT RETAINED (g)	% RETAINED	% TOTAL PASSING
GRAVEL	3"	0.0	0.0%	100.0%
	1"	0.0	0.0%	100.0%
	3/4"	0.0	0.0%	100.0%
	1/2"	0.0	0.0%	100.0%
	3/8"	7.3	4.1%	95.9%
	#4	10.6	6.0%	89.9%
SAND	#8	9.5	5.3%	84.6%
	#10	3.0	1.7%	82.9%
	#16	10.0	5.6%	77.3%
	#40	58.3	32.8%	44.5%
	#50	25.7	14.5%	30.0%
	#100	37.1	20.9%	9.2%
	#200	5.3	3.0%	6.2%
	pan	1.0	6.2%	0.0%

HYDROMETER

˚C	T min	R	δR	R - δR	L	L / T	K	D	P%
	0.5								
	1								
	2								
	5								
	15								
	30								
	60								
	120								
	240								
	480								
	1440								

WATER CONTENT		
CAN No:	PTA1	
Soil wet + Tare	216.2	
Soil dry + Tare	192.0	
Weight water	24.2	
Tare	14.2	
Weight dry soil	177.8	
W%	13.6	

$$P\% = \frac{(R - \delta R)\, a}{Ms} \times 100$$

Dmm = K √L/T   L is function of (R + meniscus) R'

Dr = ____ ☐ measured ☐ estimated

a = ____

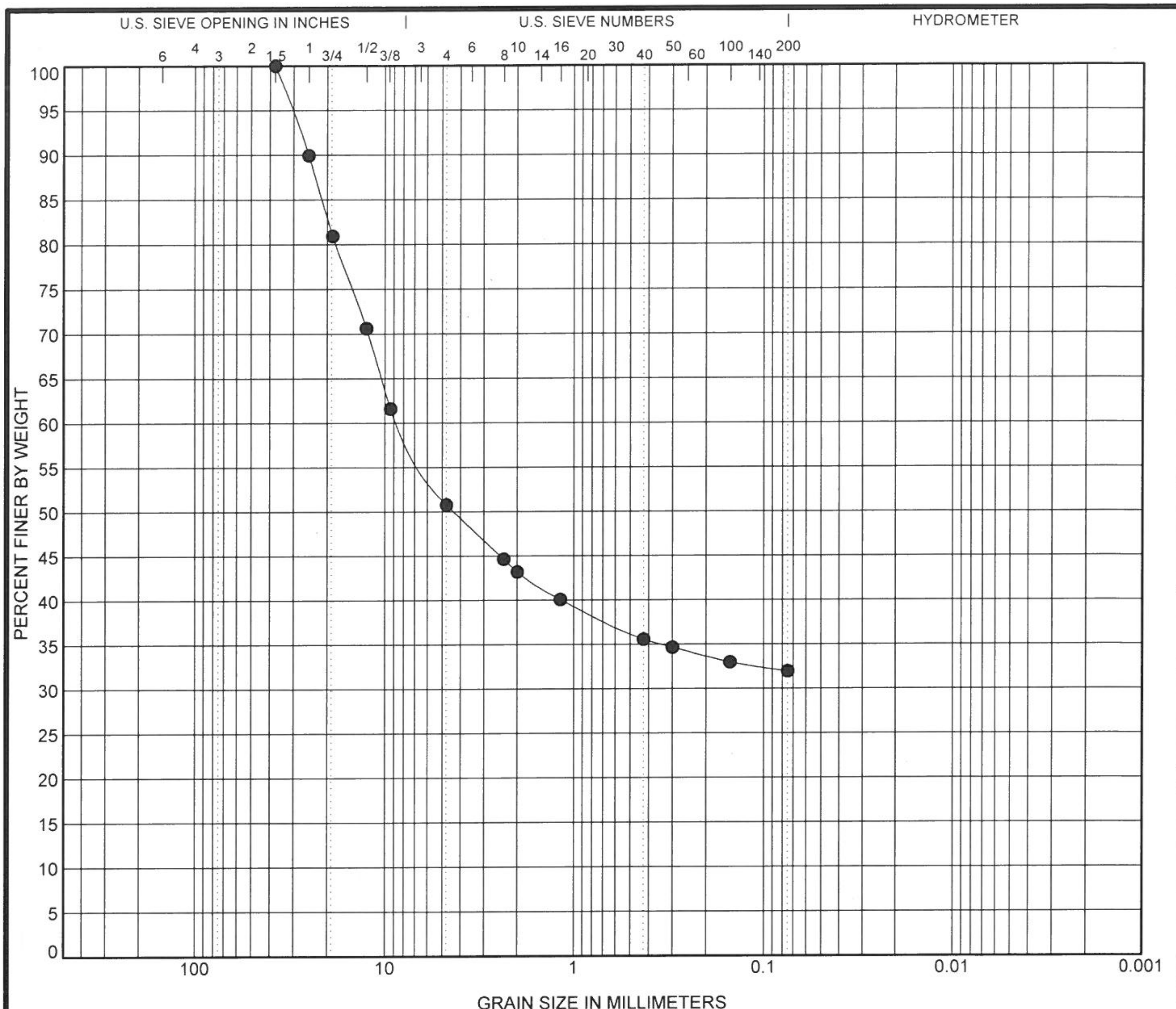
DISPERSING AGENT: ____

CONCENTRATION: ____ g/L

REMARKS

PERFORMED BY: Dan Kribs DATE: 1/10/2008

CHECKED BY: [signature] DATE: 1/10/08



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	B-3	23.5	(GM) Silty GRAVEL with fine to coarse sand, brown.					

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-3	23.5	37.5	8.471			49.2	18.8	32.0	

GRAIN SIZE D020036 YMCA.GPJ CRA PLYMOUTH.GDT 2/4/08

INSPECSOL ENGINEERING INC.
MEMBER OF THE CRA FAMILY OF COMPANIES

GRAIN SIZE DISTRIBUTION

Project Name: Former YMCA Development Project
Project Number: D020036
Client: Haengel & Associates Engineering, Inc.
Location: Ann Arbor, MI

Test by: DA **Checked by:** [illegible]

INSPECSOL ENGINEERING INC.
MEMBER OF THE CRA FAMILY OF COMPANIES

PARTICLE-SIZE ANALYSIS OF SOILS (ASTM D 422-63)

CLIENT: Haengel & Associates Engineering, Inc. LAB NO: 115
PROJECT / SITE: YMCA Development Project PROJECT NO: D020036

BORING NO: B-3 SAMPLE NO: S-7 DEPTH: 23.5' - 25'
SOIL DESCRIPTION: (GP-GM) Silty GRAVEL with fine to coarse sand, brown.

APPARATUS:
Balance No: ______ Sieve No: ______ Hydrometer No: ______
Oven No: ______ 1000ml Cylinder No.: ______ Hydrometer Jar No: ______

SIEVING

TOTAL DRY WEIGHT: 348.3 g
Weight after washing: 238 g
Loss by wash: 110.3 g

DRY ______
HUMID ______
% PASSING #10 ______

	SIEVE SIZE	WEIGHT RETAINED (g)	% RETAINED	% TOTAL PASSING
GRAVEL	3"	0.0	0.0%	100.0%
	1"	35.0	10.0%	90.0%
	3/4"	31.4	9.0%	80.9%
	1/2"	35.9	10.3%	70.6%
	3/8"	31.4	9.0%	61.6%
	#4	37.7	10.8%	50.8%
SAND	#8	21.4	6.1%	44.6%
	#10	5.0	1.4%	43.2%
	#16	10.9	3.1%	40.1%
	#40	15.7	4.5%	35.6%
	#50	3.2	0.9%	34.7%
	#100	5.8	1.7%	33.0%
	#200	3.6	1.0%	32.0%
	pan	1.0	32.0%	0.0%

HYDROMETER

°C	T min	R	δR	R - δR	L	L / T	K	D	P%
	0.5								
	1								
	2								
	5								
	15								
	30								
	60								
	120								
	240								
	480								
	1440								

WATER CONTENT		
CAN No:	A41	
Soil wet + Tare	447.4	
Soil dry + Tare	427.9	
Weight water	19.5	
Tare	179.6	
Weight dry soil	248.3	
W%	7.9	

$$P\% = \frac{(R - \delta R)\, a}{Ms} \times 100$$

$Dmm = K \sqrt{L/T}$ L is function of (R + meniscus) R¹

Dr = ______ ☐ measured ☐ estimated

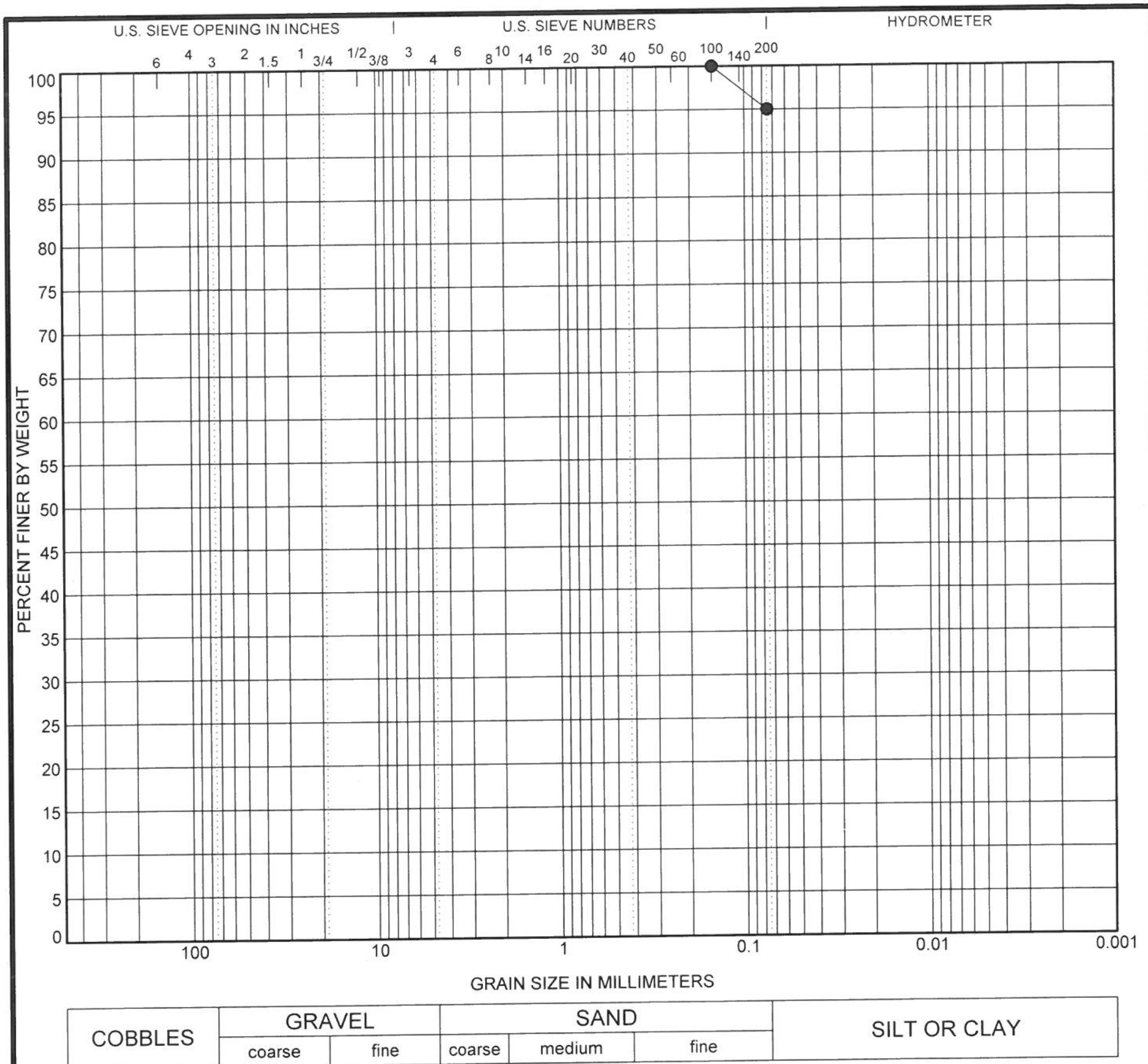
a = ______

DISPERSING AGENT: ______

CONCENTRATION: ______ g/L

REMARKS

PERFORMED BY: Dan Kribs DATE: 1/9/2008
CHECKED BY: [signature] DATE: 1/9/08



Specimen Identification		Classification	LL	PL	PI	Cc	Cu
● B-3	88.5	(ML) Clayey SILT, compact, gray, moist.					

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● B-3	88.5	0.15				0.0	5.0	95.0	

INSPECSOL ENGINEERING INC.
MEMBER OF THE CRA FAMILY OF COMPANIES

GRAIN SIZE DISTRIBUTION

Project Name: Former YMCA Development Project
Project Number: D020036
Client: Haengel & Associates Engineering, Inc.
Location: Ann Arbor, MI

GRAIN SIZE D020036 YMCA.GPJ CRA PLYMOUTH.GDT 2/4/08

Test by: DK Checked by: [signature]

INSPECSOL ENGINEERING INC.
MEMBER OF THE CRA FAMILY OF COMPANIES

LOSS BY WASH

PARTICLE-SIZE ANALYSIS OF SOILS (ASTM D 422-63)

CLIENT: Haengel & Associates Engineering, Inc. LAB NO: 115
PROJECT / SITE: YMCA Development Project PROJECT NO: D020036

BORING NO: B-3 SAMPLE NO: S-17 DEPTH: 88.5' - 90'
SOIL DESCRIPTION: (ML) Clayey SILT, compact, gray, moist.

APPARATUS:
Balance No: ______ Sieve No: ______ Hydrometer No: ______
Oven No: ______ 1000ml Cylinder No.: ______ Hydrometer Jar No: ______

SIEVING

TOTAL DRY WEIGHT: 142.4 g
Weight after washing: 8.1 g
Loss by wash: 134.3 g

DRY ______
HUMID ______
% PASSING #10 ______

	SIEVE SIZE	WEIGHT RETAINED (g)	% RETAINED	% TOTAL PASSING
GRAVEL	3"	0.0	0.0%	100.0%
	1"	0.0	0.0%	100.0%
	3/4"	0.0	0.0%	100.0%
	1/2"	0.0	0.0%	100.0%
	3/8"		0.0%	100.0%
	#4		0.0%	100.0%
SAND	#8		0.0%	100.0%
	#10		0.0%	100.0%
	#16		0.0%	100.0%
	#40		0.0%	100.0%
	#50		0.0%	100.0%
	#100		0.0%	100.0%
	#200	7.1	5.0%	95.0%
	pan	1.0	95.0%	0.0%

HYDROMETER

°C	T min	R	δR	R - δR	L	L / T	K	D	P%
	0.5								
	1								
	2								
	5								
	15								
	30								
	60								
	120								
	240								
	480								
	1440								

WATER CONTENT		
CAN No:	PT10	
Soil wet + Tare	193.8	
Soil dry + Tare	158.7	
Weight water	35.1	
Tare	16.3	
Weight dry soil	142.4	
W%	24.6	

$$P\% = \frac{(R - \delta R)\, a}{Ms} \times 100$$

$D_{mm} = K \sqrt{L/T}$ L is function of (R + meniscus) R'

Dr = ______ ☐ measured ☐ estimated

a = ______

DISPERSING AGENT: ______

CONCENTRATION: ______ g/L

REMARKS

PERFORMED BY: Dan Kribs DATE: 1/10/208
CHECKED BY: [signature] DATE: 1/10/98

3

PRESSUREMETER TEST RESULTS

Conditions For Testing With GRL's Loading Devices
January 2003

1. GRL Engineers, Inc. is a professional engineering consulting firm, specialized in the testing of deep foundations. One of its services is load testing of both driven piles and bored piles. For bored piles, often there is no convenient loading device available on site in contrast to driven piles where the pile driving hammer normally suffices. However, even for driven piles is it sometimes necessary to bring a large impact mass to the site than needed for the pile installation. For that reason, GRL has prepared several different systems with several different drop masses. Each system consists of a **guide** and a **ram**. A **free release device** is also included.

2. GRL can arrange to have the loading devices **transported** to the site and back to their storage location.

3. GRL can give **advice** as to the operation of the loading device, however, GRL is not in a position to actually handle heavy loads.

4. The client must arrange to have the **piles prepared with a clean, smooth and even top surface**. It has proven advantageous to extend the shaft by slightly more than one (1) pile diameter using a thin steel shell (casing) as a form, with good quality concrete. Windows of 8x8 inches should be cut into or an 8 inch circumferential bottom strip removed from this **steel shell** for sensor attachment to the concrete (requires a laborer and **torch**). The thickness of the shell may be as thin as 1/4 inch. The shell provides external reinforcement and a smooth surface for sensor installation. Also, this extension is easily cast from uncontaminated concrete. Under exceptional circumstances is it acceptable to excavate the pile top to a depth of at least one (1) pile diameter plus 6 inches with a width of the trench of at least 18 inches.

5. The client must provide for a **crane, crane operator and labor** that can help with the torch cutting of the windows in the steel shell, leveling of the ground around the pile, setting up blocking for the frame legs, helping with crane rigging, and climbing the guide frame for setting up the loop cutter and load transfer pin. For testing with the APPLE, the crane must be capable of lifting a load equal to the ram weight on **one line** plus the guide frame (4 tons) on a **second line**. For testing with the 3.3 ton hammer, line capacities have to be at least 4 tons each (see also operation information).

6. The client must arrange that there is an **even and firm working surface** for both crane and loading device. For APPLE testing an area of approximately 8x8 feet (or 6 ft radius around the center of the pile) must provide a stable support surface for the loading device around the pile. There should be sufficient **blocking material** on the site to spread the load of the four guide frame legs (each APPLE leg carries approximately 1 ton plus 1/4 of the ram weight.)

7. GRL provides the electronic testing equipment (PDA) and normally two experienced test engineers. GRL collects the data, analyzes the data and writes a report. However, GRL will not take responsibility for rigging and crane operation.

OPERATION OF THE APPLE

March 2003

NOTE: Operation of the APPLE involves heavy loads and should only be conducted by personnel experienced in crane operation, load rigging and construction procedures. All OSHA rules shall be observed.

Components of the system

The dynamic loading system, APPLE, consists of

- Frame with top cross beam
- Ram section(s)
- Ram pick-up post(s) with bottom plate
- Ram top plate and wedges
- Hole bar
- Load transfer pin
- Hydraulic shear
- Hydraulic pump and hose
- Severable loops (one for each impact)

The ram may consist of several units which are held together by the pick-up post(s) using a top plate and wedges.

Required crane

Assembly and operation of the APPLE requires a 2 line crane with a capacity (at 25 ft radius) which is equal to ram weight plus 5 tons. A minimum boom height of 50 ft is also needed at 25 ft radius. One line has to have a capacity in excess of the ram weight; the second line has to be capable of lifting at least 4 tons. A hydraulic crane is satisfactory.

The crane is used (a) for the unloading, assembly and loading of the system and (b) for ram lifting prior to ram release by severing one of the loops. The ram weight is slowly set onto the frame prior to the ram release. In this way it is avoided that the ram is suddenly released from the crane.

It is important that the frame be leveled prior to ram lifting and release. For this reason a level and competent working surface must be available around the test shaft. This area should be of at least 6 ft radius around the center of the test shaft. The four legs of the frame have to support the weight of the frame and ram. Thus, each leg has to support approximately 1/4 of the ram plus frame weight. Sufficient blocking has to be provided so that the frame will not experience undue settlements prior to ram release and also for leveling.

Assembly

For single unit rams, the ram has to be assembled with the post and securely wedged to the post. Four shackles and slings (provided) are then attached to the top of the four frame posts, and hooked to the auxiliary line of the crane. The frame is picked up and the frame is then threaded over the ram.

For rams, which consist of several units, the bottom ram section has to be first set in the vertical position over the center post. Two eye-bolts are then screwed into the 1-1/4 inch holes on top of the second ram section. The second section is lifted and set over the center post. The eye bolts are then removed. If a third section is used, this process is repeated. After the last ram section has been set, the round top plate is threaded over the top bar and then the two-part wedge is tightened against the top plate and through the center posts slot. In this way, the ram sections are pre-compressed.

After the ram has been assembled and wedged, the hole bar is inserted into the frame though its top cross beam. The hole bar is connected to the top of the ram post with three shackles. The top of the hole bar is then hooked to the crane's main block.

The ram and frame assembly is then picked up and set over the pile to be tested. First, care has to be taken that ram and pile center are well aligned. Then the frame must be plumbed. All four frame legs have to be supported by blocks.

The center shackle connecting hole bar to post is removed.

Operation

A wire loop with a rating that corresponds to the ram weight is installed between the two shackles at the top of the center post and the bottom of the hole bar.

The hydraulic cutter is attached to the wire loop and secured loosely to the frame so that it will not fall to the ground after wire loop severing.

The hole bar-ram assembly is slowly lifted a short distance.

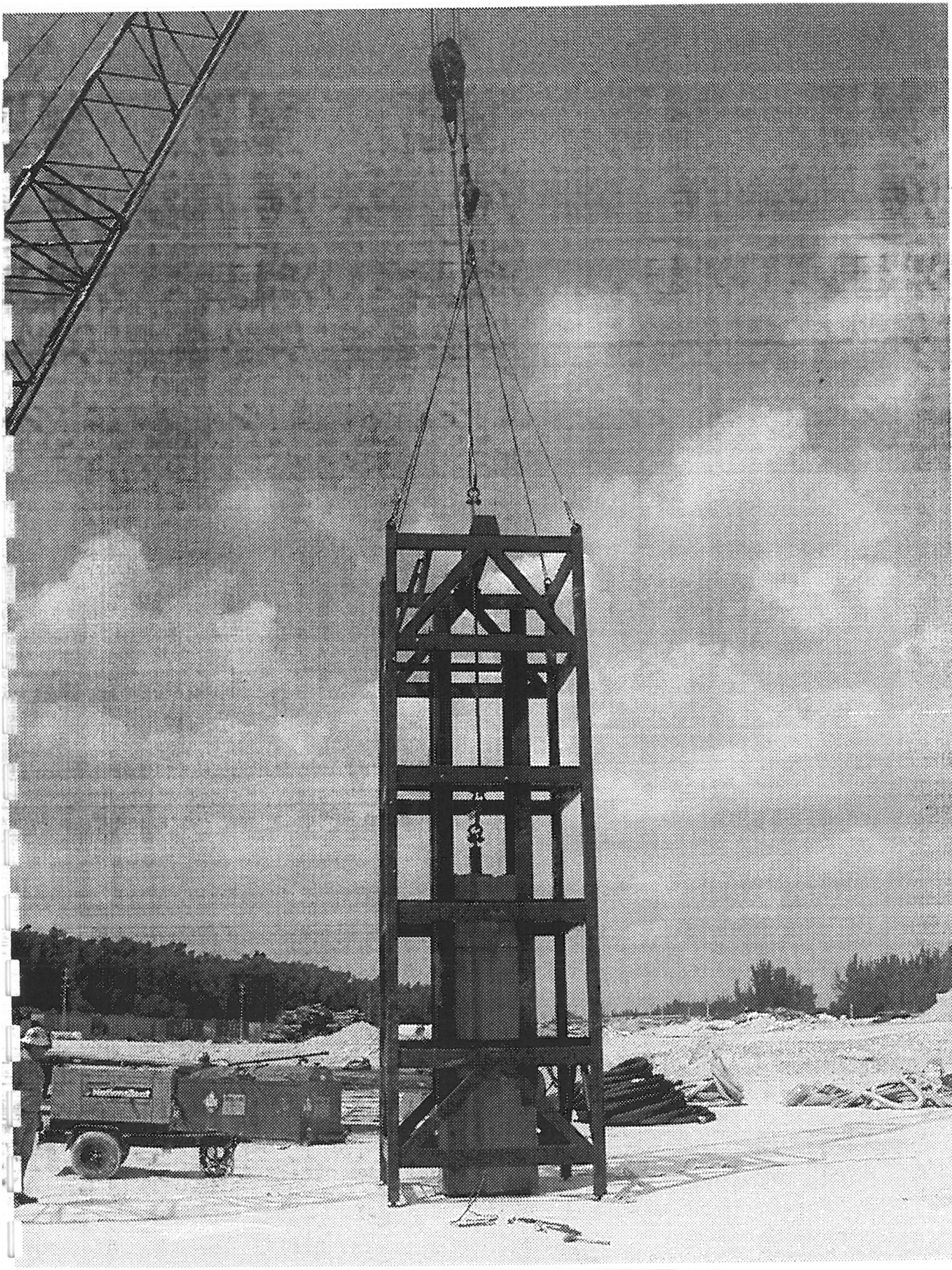
Stay clear of frame and assure that it legs are properly supported: **Observe that the frame remains plumb.**

If necessary one or more plywood cushion sheets are inserted between ram and pile top. (**Caution: keep clear of ram**).

After climbing to the top of the frame (**use safety belt**), an operator inserts the load transfer pin through the hole in the hole bar which is located just above the cross beam.

The crane releases the ram load thereby transferring the ram weight from the crane to top cross bar and frame.

The hydraulic cutter is activated severing the wire loop and causing ram release.



4

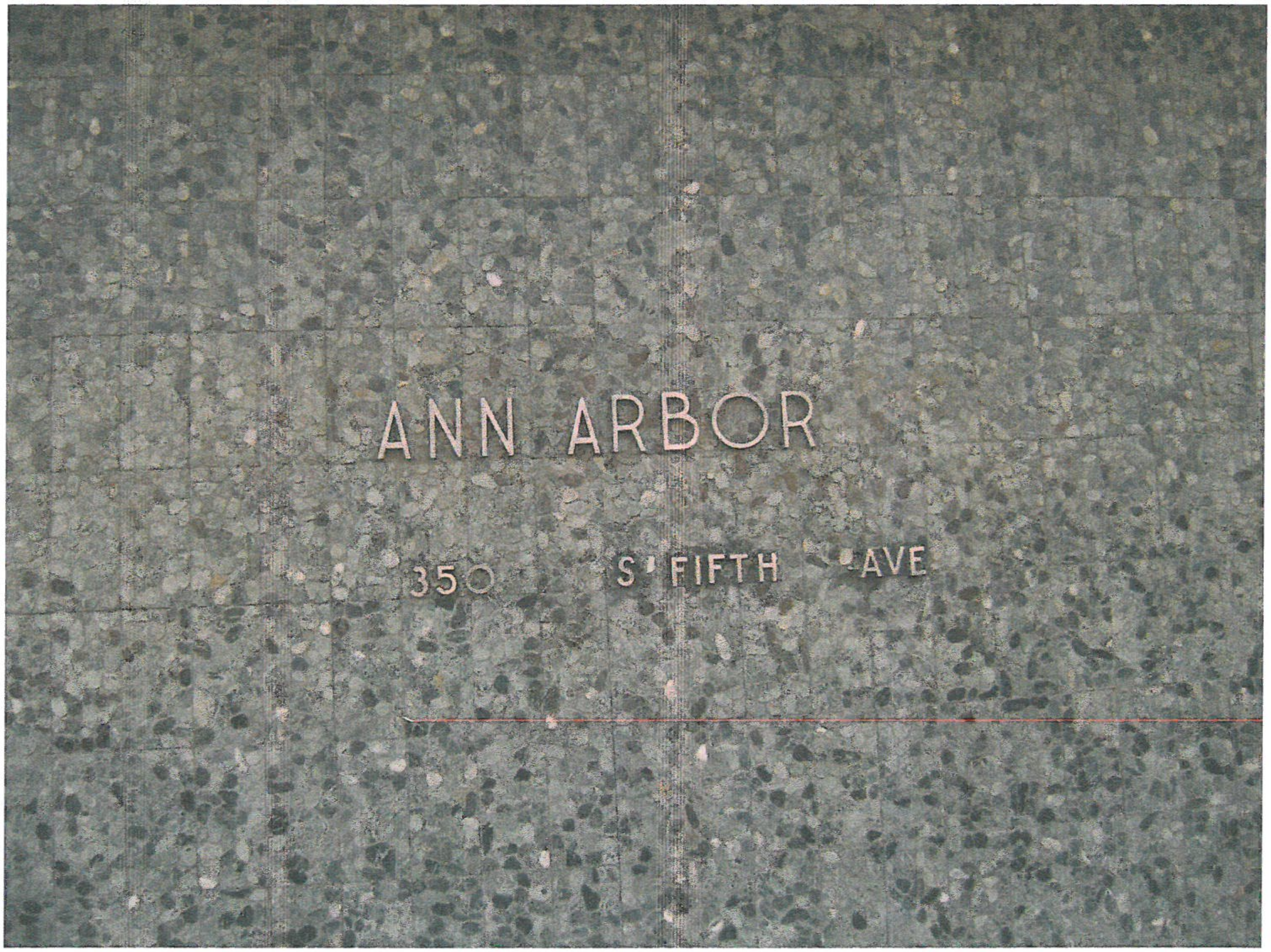
SITE PICTURES

Geotechnical
Environmental
Construction

42030 Koppernick Rd., Suite 318
Canton, Michigan 48187
734-455-9771
Fax: 734-455-9774
Email: haei@earthlink.net

Project Name: **Old YMCA Site**
350 S. Fifth Avenue, Ann Arbor, Michigan

Project No.: **H08-803-G**



December, 2007

Geotechnical

Environmental

Construction

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Geotechnical

Environmental

Construction

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Project Name: **Old YMCA Site**
350 S. Fifth Avenue, Ann Arbor, Michigan

Project No.: **H08-803-G**

December, 2007

HAE

Geotechnical

Environmental

Construction

42030 Koppernick Rd., Suite 318
Canton, Michigan 48187
734-455-9771
Fax: 734-455-9774
Email: haei@earthlink.net

Project Name: **Old YMCA Site**
350 S. Fifth Avenue, Ann Arbor, Michigan

Project No.: **H08-803-G**

December, 2007

HAE

Geotechnical

Environmental

Construction

42030 Koppernick Rd., Suite 318
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Project Name: **Old YMCA Site**
350 S. Fifth Avenue, Ann Arbor, Michigan

Project No.: **H08-803-G**

December, 2007