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Stones Throw Housing Development at Fletcher Lane and Territorial Road in Hassan Township, Minnesota

Hassan Mainstreet, LLC
c/o The Beard Group
Hopkins, Minnesota

STS Project 200703926

October 19, 2007

THE INFRASTRUCTURE IMPERATIVE

STS CONSULTANTS

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October 19, 2007

Hassan Mainstreet, LLC
c/o The Beard Group
750 - 2nd Street NE, Suite 100
Hopkins, MN 55343

Attn: Mr. Paul Gamst

Re: Subsurface Exploration and Geotechnical Engineering Analysis for the Proposed Stones Throw Housing Development at Fletcher Lane (County Road 116) and Territorial Road in Hassan Township, Minnesota; STS Project 200703926

Dear Mr. Gamst:

STS performed a Phase 1 subsurface exploration at this site and submitted a geotechnical engineering report dated May 22, 2006 (STS Project 200603005). That exploration found deposits of peat in the west and west-central portions of the property. We have since performed a Phase 2 exploration which better defines the outlines of the peat deposits. The attached report contains the logs of 28 Phase 1 borings and 41 Phase 2 borings, an evaluation of the conditions encountered in the borings, and our recommendations for site preparation for building construction, suitable foundation type, allowable soil bearing pressure for footing design, lateral stresses on subgrade walls, parameters for the internal roadway design, and other geotechnical related design and construction considerations.

We appreciate the opportunity to work with you and Schoell & Madson on this project. If you have any questions about our findings or recommendations, please call us at 763-315-6300. To arrange for our testing services during the earthwork and construction phase of this project, please call either of the undersigned at the same phone number.

Sincerely,

STS CONSULTANTS, LTD.

Mervyn Mindess

Mervyn Mindess, P.E.
Senior Project Engineer

James H. Overtoom, P.E.
Principal Engineer

MM/dn
Encs.
cc: Mr. John Gumbrill - Building Construction Management, Inc.
Mr. Jay Hill - Schoell & Madson, Inc.

I HEREBY CERTIFY THAT I AM A PROFESSIONAL ENGINEER REGISTERED UNDER THE LAWS OF THE STATE OF MINNESOTA, AND THAT THIS REPORT WAS PREPARED BY ME OR UNDER MY DIRECT SUPERVISION.

Signed Mervyn Mindess Registration No. 8435
Mervyn Mindess, P.E.
Date Oct. 19/07

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1.0 PROJECT OVERVIEW

1.1 Project Description

A planned unit development will be constructed on this property. The development will include single family dwellings, townhomes, and commercial properties at the north end, fronting on the major roads.

The single family dwellings will have basements or partial basements, masonry or cast-in-place concrete foundation walls to grade or first floor level, with wood frame construction above. The townhomes may be slab-on-grade or may have basements. Construction would be similar to that for the houses. The commercial buildings will be one to two story slab-on-grade structures.

We estimate that structural loads on the residential building wall footings will be in the range of 1.2 to 2.2 kips per lineal foot, with interior column loads in houses of 25 to 55 kips, and interior bearing wall loads in townhomes of about 1.5 to 1.8 kips per lineal foot. The commercial buildings would be framed with masonry bearing walls with brick veneer on the exterior, or with steel studs and EIFS (exterior insulation finishing system), with an open web steel joist structural system to support the roof. We estimate loads of 2 to 4 kips per lineal foot on the bearing walls with interior column footing loads of 40 to 60 kips.

An internal roadway system with underground utility lines will be constructed to serve the development. It is likely that there will be cut at the north end of the site and fill at the south end.

1.2 Project Scope and Purpose

Our services for Phase 2 of this project were performed in accordance with our proposal dated July 27, 2007. This work was authorized by Mr. Paul Gamst on July 30, 2007.

The purposes of this phase of the exploration are to:

- Drill and sample 41 additional soil borings (B-30 to B-70) to 15 to 20 foot depths. We encountered peat, loose or soft soils in some of the borings, and extended them deeper.
- Describe the soil and groundwater conditions encountered in all 70 borings.
- Attempt to delineate the extent of the peat and organic soils, to facilitate the final planning.
- Characterize the subsurface conditions with respect to the site geology and the proposed construction.
- Analyze the available subsurface information which is applicable to this project.
- Present recommendations for design of foundations and floor slabs.
- Discuss the construction considerations related to earthwork and foundations.

2.0 EXPLORATION AND TESTING PROCEDURES

2.1 Boring Layout and Soil Sampling Procedures

Schoell & Madson requested the number of borings and selected the boring locations. STS recommended the boring depths. The boring locations were staked for us by a survey crew from Schoell & Madson, Inc. who provided surface elevations to National Geodetic Vertical Datum. The approximate boring locations are shown on the Soil Boring Location Diagram in the Appendix.

We drilled the Phase 1 borings on May 3 through 9, 2006 with a CME-45 drill rig mounted on a Bombardier tracked carrier. We drilled the Phase 2 borings on August 3 to 13, 2007 with an all-terrain CME-750 drill rig. Each drill rig was operated by a two person crew. The drill crews advanced the borings using continuous flight hollow stem augers. Detailed descriptions of typical drilling procedures are included in the Appendix. Drilling methods, depths, casing usage, drill rig type, foreman, and other drilling information are indicated on the boring logs.

The drill crews sampled the soil in advance of the auger tip at 2.5 foot intervals of depth to 10 feet and at 5 foot intervals thereafter. The soil samples were obtained using a split-barrel sampler which was driven into the ground during standard penetration tests in accordance with ASTM D-1586, Standard Method of Penetration Test and Split-Barrel Sampling of Soils. An explanation of typical STS drilling and sampling procedures is presented in STS Field and Laboratory Procedures in the Appendix.

Recovered soil samples were described on field logs, containerized, and transported to our laboratory for further examination and testing. The field logs also document sample intervals, test data, observations of drilling resistance, groundwater occurrence and other pertinent conditions.

2.2 Groundwater Measurements and Borehole Abandonment

The drill crews observed the borings for free groundwater while drilling and after completion. We installed a number of PVC piezometers during the Phase 1 exploration, and an STS engineering technician measured the groundwater levels in the piezometers on two separate occasions. The crew backfilled the borings without piezometers with soil cuttings, to comply with Minnesota Department of Health regulations.

2.3 Laboratory Testing Procedures

The penetration test split-spoon samples were visually examined by a geotechnical engineer to estimate the distribution of grain sizes, plasticity, consistency, moisture condition, color, presence of lenses and seams, and apparent geologic origin. The engineer classified the soils according to type using the STS Classification System, which is closely based on the Unified Soil Classification System. A chart describing the STS Classification System is included in the Appendix. An explanation of typical laboratory procedures is presented in the Appendix.

2.4 Boring Log Procedures and Qualifications

The results of the field and laboratory observations and tests are printed on final boring logs included in the Appendix. Similar soils were grouped into the strata shown on the boring logs, and the appropriate estimated USCS classification symbols were also added. Note that the stratification depth lines between soil types on the logs are estimated based on the available data. In-situ, the transition between soil types may be distinct or gradual in either the horizontal or vertical directions. The soil conditions have been established at our specific test hole locations only. Variations in the soil stratigraphy may occur between and around the borings, the nature and extent of which would not become evident until exposed by construction excavation. These variations must be properly assessed when utilizing the information presented on the boring logs. Additional comments on boring log preparation and qualifications are contained in an Appendix sheet entitled STS Standard Boring Log Procedures.

3.0 EXPLORATION RESULTS

3.1 Site and Geology

Rush Creek crosses the south end of the site, and flows in a west to east direction. The property slopes from the north down toward Rush Creek to the south. There is approximately an 82 foot maximum difference in elevation among the borings, between the high point at boring B-3 (elevation 988.7 feet) and the low point at boring B-62 (elevation 906.6 feet). There is also a wetland in the west-central portion of the property and a tributary of Rush Creek drains southward through this wetland toward the creek. A portion of the site is wooded, and the lower southerly portion is used partly for sod fields.

The basic naturally occurring soil type at this site is sandy clay glacial till deposited by the Des Moines ice lobe of the Wisconsin glaciation. After this soil was formed, it was modified by fluvial action, and the upper portion now contains random seams or lenses of sand, silty sand and silt, some of which are water-bearing. Post-glacial deposits of peat and organic silt formed in low-lying poorly drained areas at the westerly portion of the site.

3.2 Soil Conditions

We found peat and organic silt in the north-central and west-central portions of the property, and the approximate extent of the peat deposits is indicated in light brown shading on the site plan. There are also a few small zones around the margins of the peat formation where the peat depth is 5 feet or less, and these are indicated in red. This is a depth which could be economically excavated and replaced with new compacted fill. The most extreme peat thicknesses occurred in borings B-6, B-43, B-52, B-60 and B-62, where the peat and organic silt depths range from 17 to 39 feet in total thickness.

We found fill in borings B-35, B-39 and B-69, to depths of 10.6, 6.5, and 7.6 feet, respectively. The fill is basically lean sandy clay, but is contaminated with topsoil at several locations. It is considered marginal at best, and would probably have to be at least partially removed from below any structure.

In the other borings, outside the peat and fill areas, we found from 0.4 to 2 feet of organic topsoil with roots.

Below the fill, peat and organic silt, and topsoil, we found mostly low plasticity sandy clay glacial till. At some locations, namely borings B-55, B-56, B-57 and B-65, there are actually layers of sand or silty sand below the topsoil or peat. This sand tends to be loose, but it is compactable, and it could remain under structures.

Standard penetration N-values in the sandy clay, exclusive of the organic clayey silt, range from 4 to 2 to 25, and tend to increase with increasing depth. The uppermost few feet of this stratum is soft, particularly where it directly underlies the peat, but the major lower portion is firm to very stiff in consistency. Natural moisture contents of the sandy clay generally range from 20% to 34%.

The estimated optimum moisture content of this soil that would be determined by a Proctor compaction test is about 15% to 17%. Thus it can be seen that the native clay is substantially wetter than optimum, and will require drying to facilitate proper compaction.

3.3 Groundwater Conditions

We observed free groundwater in 39 of our 70 borings, at depths of 2.5 to 21.6 feet below ground surface, corresponding to elevations 886.6 to 971.5 feet. During the Phase 1 exploration, we installed 13 piezometers. Observed groundwater levels in the piezometers, even after bailing, were at depths of 2.4 to 10.1 feet below ground surface, corresponding to elevations 904.6 to 985.9 feet.

A large wetland approximately one mile south of the site has a normal summer water elevation less than 920 feet. The section of Rush Creek which traverses the south end of the property has normal summer water elevations of about 910 feet at the west property line and less than 910 feet at the east property line, with a gradient from west to east. French Lake is about one mile northeast of the northeast property corner, with an average summer water elevation of 904 feet.

We were informed by the City of Maple Grove Engineering Department that the design 100-year flood elevations of Rush Creek, as obtained from the FEMA map, are 914.5 feet at the crossing under Fletcher Lane, 913.0 feet at the crossing under Brockton Lane, and 912.0 feet at the crossing under Interstate I-94.

The 31 borings which were dry immediately after completion of drilling penetrated mostly through clay soils, with an absence of silt or sand seams. Thus these holes were dry. The borings where we did penetrate silt and sand seams are the ones where we observed free groundwater.

Our interpretation of these data is that all groundwater levels which we recorded that were higher than elevation 915 feet represent localized perched groundwater conditions. The groundwater is collected within pervious sand lenses within the clay till profile, and is retarded from further downward percolation by the underlying low permeability sandy clay. The perched groundwater is not insignificant, and will be encountered in some excavations. However it can be controlled by pumping from sump pits or by gravity drainage to Rush Creek. For design purposes, the hydrostatic groundwater elevations at this site should be taken to be 925 feet or less at the north end of the property, grading to 915 feet or less at the south end adjacent to Rush Creek.

4.0 ANALYSIS AND RECOMMENDATIONS

4.1 Discussion

In addition to the conventional grading that will be required at this site, zones of upper soft, compressible soil should also be removed and replaced with controlled compacted fill. The individual buildings on this site can then be supported on spread footing foundations.

The silty and clayey soils at this site constitute a poor pavement subgrade. Thus a thicker pavement design will be required for good performance.

4.2 Site Preparation for Buildings

Because of the large area of the site and the limited number of borings, additional site specific borings or test pit exploration programs would be required for us to provide more detailed recommendations regarding site development.

As stated previously, upper soft soils must be removed prior to construction or prior to placement of new fill. The following table presents our recommended depths of excavation for the borings outside of the deep peat area:

Soil Boring No.	Surface Elevation - ft.	Recommended Minimum Depth of Cut - ft.	Approximate Elevation of Excavation Base - ft.
30	978.2	2	976
1	964.0	1	963
31	971.8	3	969
32	978.3	2	976
33	968.1	2	966
2	981.0	2	979
34	950.9	4.5	946.5
37	955.6	0.5	955
36	986.0	1	985
3	988.7	1	987.5
35	973.3	6.5	967
38	926.9	0.5	926.5

Soil Boring No.	Surface Elevation - ft.	Recommended Minimum Depth of Cut - ft.	Approximate Elevation of Excavation Base - ft.
5	939.7	1.5	938
4	915.0	4	911
40	910.1	3	907
41	921.0	1	920
44	935.8	5	931
8	944.0	1	943
7	908.7	3	905.5
50	924.7	0.5	924
9	933.2	1	932
14	908.2	1	907
12	923.5	0.5	923
54	907.6	4	903.5
11	907.6	1.5	906
55	930.6	1	929.5
56	915.0	0.5	914.5
58	916.3	0.5	916
13	918.3	1	917
15	908.2	1	907
16	917.3	0.5	917
17	934.5	1	933.5
18	941.3	0.5	940.5
23	916.3	1.5	915
21	913.0	1.5	911.5
19	925.5	1	924.5
20	907.9	1.5	906.5
65	910.4	2	908.5
25	908.9	1	908
29	917.9	4	914
66	919.9	2	918
27	917.8	1	917
26	924.1	1.5	922.5
67	923.8	1	923
68	921.4	1	920.5
28	916.8	1.5	915.5
69	921.3	5	916.5
70	912.1	1.5	910.5

The soil excavated based on the above recommendations is not usable as structural fill. It can be used for landscaping purposes or may have to be hauled off-site.

The on-site, inorganic, sandy and clayey silt/sandy clay is reusable as structural fill. This soil should be moisture conditioned to within ±2% of the optimum moisture content determined from a Standard Proctor compaction test. It should be placed in 6 to 8 inch loose lifts and compacted with vibratory sheepsfoot rolling equipment to at least 95% of the maximum Standard Proctor dry density, ASTM D-698.

Most of the fine grained inorganic soils at this site have relatively high natural moisture contents. Thus the earthwork contractor should allow for disking, spreading and drying to lower the moisture content before fill placement.

The following applies to structural fill in building pads only. Special planning considerations should be given to areas where the thickness of structural clay fill exceeds 10 feet. The reason for this is that such fills are susceptible to a process known as hydro-compression or hydro-consolidation. This can reduce the height of the fill by 1 to 2 percent, corresponding to settlements of a 10 foot fill in the range of 1-1/4 to 2-1/2 inches. In deep fill areas, there appear to be three possible alternatives:

- The fill in these areas should be compacted to a minimum of 98% of the maximum Standard Proctor dry density, with very careful moisture content control during placement. This is very difficult to do.
- Structures should not be built on these lots for a period of at least one year after completion of fill placement. This will permit the natural hydro-compression process to occur prior to building construction.
- If the footings are set deeper, so that they are not underlain by more than 10 feet of fill, and if cast-in-place concrete foundation walls are used in conjunction with the lower footing, then construction could proceed without a time delay.

4.3 Building Foundations

The individual single family dwellings and townhomes may be supported on conventional spread footings, bearing on naturally occurring stiff to very stiff sandy and clayey silt/sandy clay, or on controlled compacted silt/clay fill, or on medium dense silty and clayey sand. The footings throughout should be designed for a net allowable soil bearing pressure not to exceed 2,000 psf. Commercial buildings at the north end of the property can also be supported on spread footing foundations. If we are notified of the design floor elevation and the original ground elevation, we can advise regarding an allowable soil bearing pressure for footing design.

Perimeter building footings should be based at least 3 feet 8 inches (3.67 feet) below outside finished grade for frost protection. Unheated garage or deck footings should be based at least 4.5 feet below grade. Continuous strip footings under perimeter bearing walls should be at least 22 inches wide, and under interior bearing walls, they should be at least 16 inches wide. Individual interior column footings should bear at least 16 inches below the top of the floor slab, and should be at least 30 inches wide.

The factor of safety against shear or bearing capacity failure for this footing design would be approximately 3. We estimate that total and differential settlements corresponding to our assumed structural loads would be about 1 inch and 1/2 inch, respectively, provided the bearing soils are not frozen or disturbed at the time of construction. However as recommended in the preceding section of this report, hydro-compression settlement would have to be considered and properly dealt with.

4.4 Foundation Wall Drainage and Backfill

A perimeter drain system should be installed around each house basement or partial basement in accordance with the provisions of the 2006 International Building Code, Sections 1807.1.3, 1807.4, 1807.4.2, and 1807.4.3. The backfill around and over the drain pipe should be a free-draining granular material such as pea gravel or 1 inch crushed rock. We strongly recommend that silty or clayey soils not be used as wall backfill, except for the uppermost 12 to 18 inches. Silty and clayey backfill against the entire height of the walls would slow or stop the flow of infiltrating water to the drain pipes. This could lead to a buildup of hydrostatic pressures against the walls, and contribute to seepage through the walls.

Subgrade walls should be thoroughly braced internally prior to backfilling on the exterior. The backfill within a zone of 18 inches around the walls could consist of imported sand having less than 7% passing the No. 200 sieve. Alternatively, an easier and probably more economical solution would be to apply drainage boards against the exterior of the walls after they have been waterproofed. The drain boards must of course connect with the perimeter drain tile system. These could consist of Miradrain™ or Enkadrain™ or equivalent. If drainage boards are used, then on-site naturally occurring soils could be used as wall backfill. The wall backfill should be compacted sufficiently so that it does not settle after construction, but it should not be over-compacted because this exerts excessive lateral stresses against the walls. The exterior finished grades should be sloped positively away from the walls in all directions, to reduce groundwater infiltration into the backfill.

Basement walls backfilled in the manner described should be designed to resist the lateral pressure exerted by an equivalent fluid having a density of 46 pounds per cubic foot.

4.5 Ground Supported Floor Slabs

The recommended site preparation program will provide appropriate support for the basement, slab-on-grade, and garage floor slabs. In our opinion, a vapor barrier is not required below the ground supported slabs unless they are to be surfaced with wood flooring. In that event, a vapor barrier should be installed.

4.6 Utility Trench Backfill

4.6.1 Areas Outside of the Deep Peat Formation

On-site inorganic soils could be used as utility trench backfill. The trench backfill above the crown of the pipe should be placed at a moisture content within ±3% of the optimum moisture determined from the Standard Proctor compaction test. The trench backfill should be spread in 8 to 10 inch loose lifts and compacted to at least 95% of the maximum Standard Proctor dry density up to a level of 3 feet below design pavement subgrade, and to at least 100% of the Standard Proctor density within the uppermost 3 feet. If proper drying cannot be achieved in the field, because of weather conditions, then the backfill should be compacted to at least 98% of the zero air voids maximum density for that particular moisture content. However in no event should fill be placed which is more than 8% wetter than optimum. Inadequate compaction or use of wet clayey silt/sandy clay backfill would likely result in additional settlement and distress of the overlying pavement.

4.6.2 Areas Within the Peat Formation

Wherever the peat depth is less than 9 feet, the peat should be totally removed from below the utility lines, and replaced with pit run sand fill.

At locations were underground utility lines traverse a deep peat area, the lines should be supported on helical screw piles spanned with a pipe cradle (beam) to support the pipe. Further design review is necessary for pile supported utilities.

It is possible that the helical screw piles can be avoided in some areas. The pipe subsoil and possibly part of the backfill above the pile could be replaced with expanded polystyrene (EPS) which weights only 1.5 pounds per cubic foot. However this requires further study when the utility alignment and invert elevations are known, because EPS should only be used above the hydrostatic groundwater level.

4.7 Pavement Areas

Site preparation in pavement areas which are in a cut section should include removing topsoil and soil containing vegetation and roots. After these areas have been cut to proposed subgrade elevation, the exposed naturally occurring subgrade should be test rolled with a fully loaded dump truck. If excessive rutting is detected by the test rolling, the affected areas should be subcut and replaced with new compacted fill. Fill areas of roadways should be compacted to the same standards as the utility trench backfill described in the preceding section.

The compacted subgrade soils within most of this site will consist of clayey silt/sandy clay. We estimate that these soils have a stabilometer R-value of 10, and recommend that this be used for the pavement thickness design. There are also some limited areas where the subgrade soils will consist of silty and clayey fine to medium sand. We estimate that these areas have an R-value of 35.

We recommend that edge drains be installed behind curbs, discharging into the catch basins or to adjacent storm water detention ponds.

4.8 Construction Considerations

Good surface drainage toward Rush Creek or toward the new storm water detention ponds should be maintained throughout the work, so that the site is not vulnerable to ponding during or after a rainfall. The excavations for the underground utilities may encounter some groundwater infiltration. However this groundwater is perched and is limited in quantity. Groundwater intrusion into excavations at this site can be controlled by pumping from sump pits in the lowest portion of the excavation, or by gravity drainage to lower areas. Under no circumstances should fill or concrete be placed into standing water.

4.9 Winter Construction

The soils at this site are not conducive to winter earthwork and this should be avoided. Only unfrozen fill should be used. Placement of fill and/or foundation concrete must not be permitted on frozen soil, and the bearing soils under footings or under the floor slab should not be allowed to freeze after concrete is placed, because excessive post-construction settlement could occur as the frozen soils thaw.

4.10 Construction Safety

All excavations must comply with the requirements of OSHA 29 CFR, Part 1926, Subpart P "Excavations and Trenches". This document states that excavation safety is the responsibility of the contractor. Reference to this OSHA requirement should be included in the job specifications.

The responsibility to provide safe working conditions on this site, for earthwork, building construction, or any associated operations is solely that of the contractor. This responsibility is not borne in any manner by STS Consultants, Ltd.

4.11 Field Observation and Testing

We recommend that the earthwork and footing installations for this project be observed and tested by a geotechnical engineer or qualified engineering technician to determine if the soil and groundwater conditions encountered are consistent with those anticipated based on our exploration. Foundation subgrades should be tested to check for adequate bearing conditions. Subgrades for slabs, pavement and new structural fill should be test rolled and unsuitable areas improved. Fill placement and compaction should be monitored and tested to determine that the resulting fill conforms to specified density, strength or compressibility requirements. Structural materials should also be tested for conformance to specifications. STS would be pleased to provide the necessary field observation, monitoring and testing services during construction.

4.12 General Qualifications

This report has been prepared to aid in the evaluation of a planned unit development at this property and to assist the civil engineer in the design of this project. The scope is limited to the specific project and location described herein, and our description of the project represents our understanding of the significant aspects relevant to soils and foundations. In the event that there are any changes in the nature, design or location of the project, the opinions and recommendations contained in this report shall not be considered valid unless STS reviews these changes and modifies or verifies the recommendations of this report in writing. We recommend that we be authorized to review the project plans and specifications to determine if the recommendations contained in this report have been interpreted in accordance with our intent. Without this review, we will not be responsible for misinterpretation of our data, our analysis, and/or our recommendations, or how they are incorporated into the final design.

5.0 STANDARD OF CARE

The recommendations and opinions contained in this report are based on our professional judgment. The soil testing and geotechnical engineering services performed for this project have been conducted in a manner consistent with that level of skill and care ordinarily exercised by other members of the profession currently practicing in this area under similar budgetary and time constraints. No other warranty, express or implied, is made.