PRELIMINARY SOIL EVALUATION

for

PROPOSED MARINA PLACE CONDOMINIUMS
BAY CITY, MICHIGAN

October 15, 2001

PREPARED FOR:

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WAK PROJECT NO. 00-0881-0626

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APPENDIX

PRELIMINARY EXECUTIVE SUMMARY

The salient conclusions and recommendations of our preliminary soil evaluation for the proposed Condominium Development to be located in Bay City, Michigan are summarized as follows:

1. Deposits of variable to miscellaneous fill materials were encountered in each boring to depths ranging from about 3½ to 19 feet below the existing ground surface. These fill deposits were generally underlain by deposits of very soft to firm, organic clayey silts and/or very loose to loose, organic “river bottom” type sands which extended to depths ranging from about 17 to 33 feet below the surface. These existing fill and natural organic soil deposits are considered unsuitable for support of conventional spread footing type foundations and are expected to pose serious challenges with regard to construction of conventional grade-supported slabs, pavements and similar site improvements.

2. Only one soil boring (Boring B4) penetrated to competent bearing material for this initial phase of investigation. In this boring, hard sandy clay was encountered at a depth of about 68½ feet (approximate relative elevation 32½) below the surface and weathered clayey shale was encountered at a depth of about 77½ feet (approximate relative elevation 23½). At depths of about 80 to 82 feet below the surface (approximate relative elevation 20), the shale appeared to become relatively sound and competent. Prior to finalizing any recommendations for foundation design, we recommend at least 2 additional deep soil borings with rock coring and testing be performed to verify the extent, soundness and strength of the shale bedrock beneath this site.

3. Preliminarily, several alternative deep foundation systems appear feasible for this site including either top-driven steel (H-section and pipe) piles or auger cast piles. Where the piles are driven a few feet into the competent shale bearing stratum encountered at depths of about 80 to 82 feet below the surface (approximate relative elevation 20), we anticipate H-piles with nominal flange widths of 12 inches could develop individual working capacities on the order of 60 to 80 tons. Drive points would likely be necessary to protect the steel H-pile tips for this design capacity to be achieved. Nominal 10 inch diameter concrete-filled pipe piles could achieve 30 to 40 ton working capacities where they are driven a foot or so into the same competent shale bearing stratum. Somewhat lower capacity piles, requiring lighter sections and terminating in the hard sandy clay layer encountered at a depth of about 68½ feet (approximate relative elevation 32½) below the surface in Boring B4 may also be feasible for this project. For nominal 14 to 18 inch diameter auger cast piles, we anticipate individual pile working capacities in the range of 30 to 50 tons could be used for design where they extend a foot or so into the competent shale bearing stratum described above.

4. As previously mentioned, the existing variable to miscellaneous fill deposits and underlying very soft to firm, organic soils encountered to depths ranging from about 17 to 33 feet below the existing ground surface of the site are considered unsuitable for support of conventional slabs-on-grade, pavements and similar site improvements. However, the dewatering and disposal costs associated with complete removal and replacement of these soils, even for just the slab-on-grade areas, are judged to be prohibitive. Implementation of various soil improvement/reinforcement methods and/or ground modification techniques, such as surcharging the site, use of geotextiles, dynamic deep compaction, vibratory replacement, stone columns, chemical injection grouting, etc. may help to mitigate the risk of potential settlement, cracking and/or faulting of the floor slabs and pavements to varying degrees, most at a substantial cost, however, they will not eliminate the risk entirely. Therefore, we recommend the proposed grade level floors be structurally supported on a

series of grade beams which, in turn, are supported on the previously recommended deep building foundation system.

5. Realizing somewhat variable settlements and/or cracking of the pavements may be tolerable, we anticipate very limited modifications to the existing pavement subgrade soils will be made with the expectation that reduced design life and increased annual maintenance may occur. To help mitigate the costs associated with premature pavement distress, consideration might be given to delaying placement of the wearing course layer to allow time for some of the settlement/distress to occur. Use of a woven geotextile fabric or geogrids in the design pavement section could also help mitigate some of the pavement distress. Consideration should also be given to minimizing use of grade supported site improvements to the extent practical. In this regard, it may prudent to use berms as opposed to retaining walls and treated wood decking in lieu of concrete walks. Similarly, swimming pools, tennis courts, and similar types of amenities should be located on structurally supported floors or deep foundations to avoid future problems associated with long term settlement. Additional comments relative to preparing subgrades and mitigating potential problems associated with differential settlement are included in the report along with recommended subgrade modulus values for design of bituminous pavements and grade slabs.

6. Considering the close proximity of the Saginaw River and its potential to fluctuate by several feet or more on a seasonal basis, we recommend appropriate design considerations be provided to prevent or resist any potential hydrostatic uplift forces acting on the bottom of the swimming pool when it is emptied. Recommendations in this regard are included in the accompanying report.

7. We do not anticipate any serious geotechnical problems associated with constructing steel pipe pile, steel H-pile or auger-cast pile foundation systems as described in our report. Considering the somewhat variable character of the existing fill deposit encountered at this site, some problems with drilling/driving piles through the existing fill deposits should be anticipated. Although not apparent in our soil borings, it is possible some large pieces of debris or existing buried concrete structures may obstruct the augering/driving operations making it necessary to pre-excavate through the fill deposit to install the piles in some areas of the site. Due to the hard nature of the bedrock bearing stratum encountered at this site, some difficulty in augering/driving foundations into these materials should also be anticipated. We anticipate water accumulations resulting from precipitation, surface runoff or groundwater seepage in the pile cap and/or grade beam excavations should be manageable using standard sump pit and filtered pumping procedures. For deeper excavations extending several feet below the water table, estimated to be about 6 to 18 feet below the surface of the site, more substantial dewatering techniques such as pumping from slotted casings or wellpoints may be necessary to control groundwater seepage. The exposed subgrade soils are expected to be somewhat prone to disturbance by the anticipated construction operations, especially in the presence of moisture. Recommendations to help maintain stability of subgrade areas are included in the accompanying report.

The executive summary presented above is general in nature and should not be construed apart from the entire text of our report with all the preliminary conclusions, considerations and qualifications presented therein. The detailed findings of the soil evaluation and our preliminary geotechnical recommendations for design and construction of the proposed development are detailed in the accompanying report.

I. INTRODUCTION

We have completed the requested preliminary soil evaluation for the proposed Marina Place Condominiums to be located in Bay City, Michigan. This report presents the results of our preliminary evaluation, our interpretation of the soil and groundwater conditions at the soil boring locations, and our preliminary geotechnical recommendations for design and construction of the proposed development. This evaluation was performed under the direction of a registered Professional Engineer in the State of Michigan.

Written authorization to perform this preliminary soil evaluation was received from Marina Place, L.L.C. on December 27, 2000 in the form of a signed letter proposal prepared by William A. Kibbe & Associates, Inc. (WAK) and dated November 29, 2000.

A. Description of Site and Project

The subject site is situated on the west bank of the Saginaw River between the Liberty Bridge and the existing Pier 7 Marina in Bay City, Michigan. At the time of our visit, an existing 50 foot by 120 foot metal building was present in the western portion of the property and a boat slip for the Pier 7 Marina was located along the south side of the site. Although the ground surface was covered with approximately 2 feet of snow at the time of our visit, much of the remaining portions of the site appeared to be open, vacant land with some scrub and a few scattered trees present. The ground surface was somewhat mounded near the center of the site, apparently due to some previous filling activities. Based on a topographic site map provided for our use, ground surface elevations range from about 107 relative elevation near the mounded area to about 93 relative elevation adjacent to the Saginaw River and the existing slip. Surface drainage appeared to be relatively good with runoff being directed toward the river and the existing boat slip. An overhead power line was noted traversing the middle portion of the site in a north-south direction and a storm sewer crossed the northern end of the property, out-falling to the Saginaw River.

The plans for the proposed development have changed significantly since inception of this project. Initially, an irregular-shaped 7- or 8-story condominium building and a new steel sheet pile seawall for approximately 22 boat slips along the Saginaw River were proposed. The lower level of the proposed condominium building would have been a parking garage for condominium owners while a bituminous parking lot for approximately 70 additional outside parking spaces would have been provided. Some combination of steel-framing and/or reinforced concrete construction was planned for the condominium tower structure, with maximum design column loadings estimated to be on the order of 500 to 600 kips. The ground level floor system would have been a concrete slab-on-grade. Other amenities including a swimming pool and tennis courts were also being considered in the original design plans.

Due to the soil conditions encountered at the site, we understand the design concept for the project has changed from a mid-rise steel-framed tower to more conventional residential type construction.

At this time, we understand five two-story, wood framed buildings with two condominium units in each building are proposed. Structural wall loads for the proposed condominium buildings are estimated to be on the order of about 3 kips per lineal foot. Each building is proposed to be supported on a thickened mat foundation which would also serve as a concrete slab-on-grade. Although the planned finished floor elevations for the proposed condominium buildings have not been determined, we anticipate some filling of the site may be needed due to flood plain considerations. Exterior drives and parking areas would still be provided under the revised plans, however, no seawall is being proposed at this time. Plans to include a swimming pool and tennis courts are presumed to be unchanged from the original plans.

Previous preliminary recommendations for this project have been submitted when the mid-rise tower development was still being considered. It should be noted, however, the preliminary recommendations submitted herein are based on the revised conceptual plans for the 2-story residential type construction plans and they supercede any previous recommendations submitted.

B. Scope of Services

The scope of our services for this preliminary soil evaluation included the following:

1. Drill two soil borings depths to about 40 feet below the existing ground surface along the Saginaw River in the vicinity of the formerly proposed seawall; drill two soil borings to depths ranging from approximately 60 to 85 feet below the existing ground surface in the vicinity of the proposed condominium building(s); and drill one additional soil boring to a depth of approximately 10 feet below the existing ground surface in the vicinity of the proposed entrance roadway to the site.

2. Perform visual laboratory classification of each soil sample obtained from the soil borings as well as routine strength and moisture content tests on representative portions of selected cohesive soil samples.

3. Complete an engineering analysis and evaluation of the available field and laboratory data and compile a written report to include the following:

    a. A description of our field and laboratory investigation procedures.

    b. A brief description of the site and the soil and groundwater conditions observed in our soil borings.

    c. A brief description of the geology in the Bay City area and a cursory review of any mining activities which may have occurred in the immediate vicinity of the subject site.

d. A discussion of feasible types of foundation systems for support of the proposed structure(s) including preliminary recommendations for net allowable soil pressure or allowable pile capacity, anticipated depth to suitable bearing material, and estimated settlement.

e. Preliminary recommendations regarding earthwork and excavations, including general suitability and treatment of in-place soils for support of grade slabs and new pavements, placement and compaction of engineered fill, and considerations for re-using material excavated from the site for engineered fill.

f. Preliminary recommendations for soil subgrade modulus values for design of grade slabs and bituminous pavements.

g. Preliminary design considerations for in-ground swimming pool construction.

h. Preliminary recommendations regarding management of groundwater and other construction considerations relating to installation of the recommended foundation system and the anticipated earthwork operations based on the soil and groundwater conditions encountered in our soil borings.

C. General

The preliminary recommendations submitted in this report have been based on the available soil boring information and the preliminary design details for the proposed development as provided by Freiwald Staudacher Design, Inc. and Gregory Construction Company. Any revisions in the location or design details for the proposed structures should be brought to our immediate attention so we may evaluate the extent to which our preliminary recommendations may be impacted by the changes. As the final plans and specifications are developed, we should be given the opportunity to review them to verify our understanding of the anticipated project and to verify our recommendations have been properly interpreted.

The preliminary conclusions, recommendations and considerations presented herein have been based on the information obtained from 5 soil borings performed at the site of the proposed development. **Additional subsurface investigation and analysis of the site is recommended to better delineate and characterize suitable bearing strata and to help facilitate formulation of final geotechnical design recommendations for the project.** This report does not reflect changes in subsurface conditions which may occur between our boring locations. If significant variations from our reported subsurface conditions are noted during construction, we should be notified immediately to determine if modifications to our recommendations are needed.

We have strived to conduct this preliminary soil evaluation in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing under similar conditions in the locality of this project. No other warranties, implied or expressed, are made. The preliminary recommendations presented herein are intended solely for the use of Marina Place, L.L.C. and their design consultants in evaluating this site for the specific development being proposed.

II. DESCRIPTION OF INVESTIGATION PROCEDURES

A. Field Operations

Five soil borings (B1 through B5) were drilled to depths ranging from about 10 to 85 feet below the existing ground surface on the subject property for this preliminary investigation. The approximate soil boring locations are shown on the Soil Boring Location Diagram included in the Appendix of this report.

The number, depth and locations of the soil borings were determined by Marina Place, L.L.C. in consultation with William A. Kibbe & Associates, Inc (WAK). The soil boring locations were staked in the field by WAK as close to the planned locations as possible using normal taping procedures and existing landmarks as reference points. Ground surface elevations at the boring locations, as indicated on the appended Soil Boring Logs, were estimated from spot elevations shown on a topographic Base Map drawing of the site prepared by Landmark Engineering Associates, Inc. of Benton Harbor, Michigan and dated October 31, 2000.

The soil borings were performed with a truck-mounted rotary type drill rig in general accordance with the American Society for Testing and Materials (ASTM) Standards D-1586 and D-1587 for split-spoon and thin-walled tube sampling of soils, respectively. Hollow-stem continuous flight augers were used to advance the bore holes for Borings B1, B2 and B5 while rotary wash drilling methods were used to advance the remaining borings. The sampling intervals, standard penetration test results (N-values), groundwater observations and other pertinent field information are shown on the Soil Boring Logs appended to this report. The symbols and notations used on the boring logs are defined on the General Notes also appended hereto.

B. Laboratory Testing

The soil samples were sealed in labeled glass jars or Shelby Tubes in the field and returned to our laboratory where they were visually classified by an experienced soils engineer in general accordance with the Unified Soil Classification System (USCS). These descriptions appear on the appended Soil Boring Logs. A chart which describes the USCS group symbols, which appear in parentheses after the soil descriptions, is also appended.

Additionally, selected representative portions of the cohesive soil samples were subjected to moisture content, calibrated hand penetrometer and/or Torvane shear strength tests. The moisture content of a soil sample is the ratio of the weight of water in the sample to the oven-dried weight of the soil, as determined by ASTM Standard D-2216, expressed as a percentage. In the hand penetrometer test, the unconfined compressive strength of a soil sample is estimated by measuring the resistance of the soil to penetration of a calibrated spring-loaded cylinder. The capacity of the hand penetrometer is 4½ tons per square foot (tsf). The Torvane apparatus provides an estimate of the undrained shear strength of a soil sample by measuring its torsional resistance to a series of small vanes using a calibrated spring-loaded dial. The capacity of the Torvane apparatus is 1 tsf, and as such, it is usually only used to test cohesive soils with relative consistencies ranging from very soft to stiff. Results of these laboratory tests are shown on the appended Soil Boring Logs.

III. DESCRIPTION OF SUBSURFACE CONDITIONS

The soil profile description and groundwater observations discussed herein are intended to provide a brief and general summary of the typical subsurface conditions encountered at this site. For a more detailed description of the soil and groundwater conditions encountered at the respective boring locations, please refer to the appended Soil Boring Logs and Soil Boring Location Diagram. The stratification lines on the boring logs are intended to indicate a general transition between soil types and the actual transition may vary between boring locations.

A. Soil Conditions

A somewhat variable soil profile was encountered at this site. Each major component of the generalized soil profile noted in our soil borings, beginning at the ground surface, is described below:

Soil Group 1 - Surficial Topsoil FILL and Variable FILL Materials with Organics, Debris and Wood (SM-FILL, CL-FILL, SM-OL-FILL, CL-OL-FILL, OL-MISC. FILL)

At each of the boring locations, a surficial sandy topsoil layer was encountered to depths ranging from about 6 inches to 1 foot below the existing ground surface. Beneath the surficial topsoil layer at each of the soil boring locations, deposits of somewhat variable to miscellaneous fill materials were encountered to depths ranging from about 3½ to 6 feet below the existing surface in Borings

B1, B2 and B4 and to about 17½ to 19 feet below the surface in Borings B3 and B4. These variable to miscellaneous fill materials consisted primarily of granular (sandy) materials with occasional clayey layers and clay chunks and miscellaneous debris including concrete, brick, asphalt, slag, wood, metal, rubber and organics.

The granular portions of these fill materials were generally very loose to medium dense in relative density, with standard penetration resistance values (N) ranging from 2 to 20 blows per foot (bpf). The consistency of the cohesive portions of these fill materials was generally soft to stiff with recorded hand penetrometer unconfined compressive strengths ranging from less than ½ to about 1 tsf and corresponding Torvane shear strengths ranging from 0.2 to 0.5 tsf. Moisture contents within the fill deposits generally ranged from about 6 to 29 percent except where considerable amounts of wood were present and moisture contents in the range of 47 to 172 percent were recorded. Given the somewhat organic and variable nature of these fill materials, the recorded strength parameters noted above may not accurately depict the true stability of these non-engineered fill deposits.

Soil Group 2 – Natural, Very Loose to Medium Dense SAND, Organic SAND with WOOD, Organic Silty/Clayey SAND, and/or Somewhat Organic, Very Soft to Firm Clayey SILT (SP, SC-SM-OL, SM-OL and/or ML-OL)

Beneath the variable fill materials described above (Soil Group 1), "river bottom" type alluvial soils consisting of somewhat organic sands and silts with variable amounts of deteriorating wood, shells and/or plant vegetation were encountered to depths ranging from the explored depth of about 10 feet below the existing ground surface in Boring B5 to depths of about 17 to 33 feet below the existing ground surface in each of the remaining borings. Given their apparent organic content, based on visual classification procedures, these "river bottom" type soils were generally given multiple USCS group symbols which included the OL designation for low plasticity organic soils. The relative density of the predominantly granular portions of these soils ranged from very loose to medium dense with N-values ranging from 1 to 10 bpf. The consistency of the cohesive portions of these soil deposits ranged from very soft to firm with hand penetrometer unconfined compressive strengths ranging from less than ¼ tsf to ½ tsf and Torvane shear strengths ranging from 0.1 to 0.3 tsf. Corresponding natural moisture contents in these cohesive soils ranged from 12 to 76 percent with an average of about 50 percent, except where considerable amounts of wood were present and moisture contents as high as 192 percent were recorded. Based on these laboratory test results, we judge these natural "river bottom" type soils to be normally consolidated to slightly over-consolidated and highly compressible under the anticipated structural loads.

Soil Group 3 (Borings B1 through B4 Only) - Natural, Very Soft to Firm Clayey SILT and Silty CLAY (ML and CL)

Beneath the materials described above in Borings B1 to B4, deposits of clayey silt and silty clay with trace to slight amounts of organics were encountered to the explored depths of 40 to 60 feet below the surface in Borings B1, B2 and B3 and to a depth of approximately 42 feet below the surface in Boring B4. These cohesive soils were generally very soft to firm in relative consistency with hand penetrometer unconfined compressive strengths ranging from less than ¼ tsf to ¾ tsf and Torvane shear strengths ranging from 0.1 to 0.3 tsf. Corresponding natural moisture contents in these cohesive soils ranged from 38 to 63 percent with an average of about 50 percent being recorded. Based on these laboratory test results, these natural cohesive soils are judged to be normally consolidated and highly compressible under the anticipated structural loads.

Soil Group 4 (Boring B4 Only) - Natural, Medium Dense SAND (SP)

Beneath the very soft to firm clayey silts and silty clays described in Boring B4 above (Soil Group 3), deposits of fine to medium sand were encountered to a depth of approximately 58 feet below the existing ground surface. The relative density of these granular soils was generally medium dense with N-values ranging from 14 to 20 bpf.

Soil Group 5 (Boring B4 Only) - Natural, Stiff Silty to Sandy CLAY (CL)

Beneath the granular soil layers described above (Soil Group 4), layers of silty to sandy clay were encountered to a depth of about 68½ feet below the surface in Boring B4. These cohesive soils were stiff in relative consistency with hand penetrometer unconfined compressive strength values ranging from 1 to 1¾ tsf and corresponding natural moisture contents ranging from 23 to 27 percent. Based on these laboratory test results, we judge these natural cohesive soils to be slightly to moderately over-consolidated and moderately compressible under the anticipated structural loads.

Soil Group 6 (Boring B4 Only) - Natural, Hard, Silty CLAY (CL-ML)

Beneath the stiff clays described above (Soil Group 5), a layer of hard silty clay with a few sand lenses was encountered to a depth of about 77½ feet below the existing ground surface in Boring B4. Hand penetrometer unconfined compressive strength values in excess of 4½ tsf with corresponding moisture contents of 10 percent were recorded in this material. Since the recorded N-values were only in the range of 38 to 42 blows per foot in these cohesive soils, they are not judged to be indicative of the highly over-consolidated glacial till "hardpan" type soils common to the area.

Soil Group 7 (Boring B4 Only) - Hard, Clayey Shale (CL-SHALE)

Beneath the hard silty clays described above (Soil Group 6), clayey shale was encountered to the explored depth of about 85 feet below the existing ground surface in Boring B4. Hand penetrometer unconfined compressive strength values in excess of 4½ tsf with corresponding moisture contents of 13 to 15 percent were recorded in these materials. The clayey shale encountered to a depth of about 82 feet below the existing ground surface appeared to be weathered and contained occasional silty clay seams and shale fragments.

The N-values recorded in this stratum ranged from 50 blows for 1 inch of penetration to 100 blows for 4 inches of penetration. As rock coring and testing was not included in the scope of our services, we have not evaluated these materials for structural integrity or determined the rock quality designation (RQD) value.

B. Groundwater Observations

Groundwater seepage was noted during the drilling or sampling operations at depths ranging from approximately 6½ to 19 feet (approximate relative elevations 94½ to 88) below the existing ground surface in each of our soil borings. Upon completion of the drilling operations and removal of the augers from the ground, groundwater was measured at depths of approximately 26feet and 38 feet (approximate relative elevations 74 and 61) below the existing ground surface in Borings B2 and B1, respectively. Immediately following completion of Boring B5, the bore hole was observed to be collapsed at a depth of 7.5 feet (approximate relative elevation 92) with no visible evidence of groundwater seepage being noted at or above the collapsed depth of the bore hole. Since rotary wash drilling procedures were used to advance the bore holes for Borings B3 and B4, no water level measurements were recorded in these borings following their completion. The bore holes for Borings B1 and B2 were allowed to remain open for period of about 4 hours and 1 hour following their completion, respectively, at which times groundwater was measured at depths of about 20.0 feet and 7.1 feet (approximate relative elevations 79 and 93) below the existing ground surface, respectively. Since all the bore holes were backfilled shortly after completion of the drilling operations, no long term water level measurements are available from any of these soil borings. It should be noted, the bore holes for Borings B3 and B4 were plugged using a mixture of cement and bentonite grout.

In granular soils or somewhat miscellaneous fill deposits, such as those encountered at this site, a relatively short amount of time is usually required for the water level in an open bore hole to stabilize with the long term hydrostatic groundwater level. Therefore, the relatively short term groundwater observations recorded in our borings are judged to be fairly indicative of the prevailing groundwater level at this site. On this basis and from our experience in this area, and based on the elevation of the adjacent Saginaw River, we estimate the long term hydrostatic groundwater at this site was at depths of about 6 to 18 feet (approximate relative elevations 94 to 89 below the existing ground surface of the site at the time our borings were performed. This indicates the hydrostatic groundwater level is generally within the fill deposits or slightly below them at each soil boring location.

Normal variations in the depth of the prevailing groundwater level should be expected due to its gently undulating surface. Similarly, the long term hydrostatic groundwater level at this site should be expected to fluctuate with variations in precipitation, evaporation and surface runoff and with variations in the elevation of the nearby Saginaw River.

IV. DESCRIPTION OF AREA GEOLOGY AND LOCAL COAL MINING ACTIVITY

A. Area Geology

Lower Michigan is the center of a bowl-shaped, synclinal geologic structure known as the Michigan Basin. This structure consists of roughly concentric, Paleozoic sedimentary formations overlying Precambrian igneous and metamorphic basement rocks. The basin extends from Wisconsin in the west to Ontario in the east and from northern Ohio and Indiana in the south to the Canadian Shield (southern Ontario) in the north. The lithologies of the formations in the basin are predominantly sandstone, shale, limestone and evaporites.

Overburden soils lying upon the bedrock in the Bay City area are primarily associated with two different phenomena: glaciation and lacustrine deposition. The Bay City area was affected by four distinct episodes of glaciation. The most recent and the most significant in the alteration of topography in the Bay City area was the Wisconsin Glacial Episode. Evidence of advancing and receding ice sheets is visible throughout Bay County in the form of glacial end moraines. Glacial Lake Saginaw formed during the recessional phase of the Wisconsin Glacial Episode when meltwater was trapped between a large end moraine to the southwest of Saginaw, Michigan and the nose of the receding glacier to the northeast. This period of lake formation following the Wisconsin Glacial Episode is known as the Nippissing Stage. Nippissing Stage lacustrine deposition accounts for much of the overburden clay and silt deposits found in the immediate vicinity. Some of these deeper clay layers are highly over consolidated due to glacial loading during temporary readvances of the ice. Other surficial layers are slightly to considerably over consolidated due to desiccation resulting from gradual recession of the groundwater level. In addition to the lacustrine clay and silt deposits, naturally occurring isolated beach sand deposits are prevalent near the ground surface throughout the Bay City area.

B. Coal Mining Activities

The bedrock underlying the Bay City area is composed primarily of the early Pennsylvanian-aged (280 to 310 million years before present) Saginaw Formation. The Saginaw Formation is made up of sandstone, shale and limestone and contains numerous individual coal seams. Many of these coal beds were mined in the Bay County area near the turning of the 20th century. A cursory review of available coal mining maps and other related literature for the area does not reveal any coal mining activity has occurred beneath the site of the proposed development. Based on the available information, the nearest such area of mining activity is located approximately 1 mile northwest of the proposed site. On this basis, the risk of future ground subsidence in the area of the proposed development due to historical mining activities does not appear to be a concern.

V. PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

The preliminary recommendations submitted herein have been based upon the available soil boring information and the preliminary design details furnished for the proposed development. If our understanding of the project, as previously described, is inaccurate or if any significant revisions in the plans are made, they should be brought to our attention so we may determine if changes to our recommendations are required. Likewise, if significant variations in the reported subsurface conditions are encountered during construction, we should be notified immediately.

A. Foundations

1. Discussion of Foundation Alternatives

A number of foundation types have been considered for support of the proposed structure(s) to be built on this site, including conventional spread footing foundations, mat foundations, drilled shafts or caissons, and various types of piling. Additionally, we have investigated the possibility of using a GeoPier ™ soil reinforcement procedure for the site.

Spread footings are considered to be the most economical type of foundation when the soil conditions and structural loads allow them to be constructed at relatively shallow depths. Our soil borings within the planned building areas encountered about 17 to 19 feet of variable to miscellaneous fill materials overlying deposits of somewhat organic, very soft to firm cohesive soils which generally extended to depths ranging from about 32 to 33 feet below the existing ground surface.

In our opinion, the surficial fill deposit is unsuitable for support of spread footing type foundations due to the variability of its engineering and load-supporting characteristics. Furthermore, the underlying soft, organic cohesive soil deposits are considered to be highly prone to consolidation settlement, further increasing the risk of unacceptable performance of spread footing type foundations even if some or all of the existing fill materials were removed and replaced with engineered fill.

We also considered ground modification treatment of the surficial fill deposit using a process known as dynamic deep compaction to improve the density and uniformity of the fill deposits, thereby making them more suitable for support of conventional spread footings. However, this method of soil improvement is relatively ineffective with regard to reducing excessive settlement risks where organic soils are present. Furthermore, based on our preliminary laboratory test results, the underlying, normally consolidated clay deposit is judged to be at least moderately sensitive. As such, a resulting loss in strength of this material would be expected due to short term development of excess pore water pressures and remolding of the clay during the dynamic loading process. For these reasons, we do not believe the combination of dynamic deep compaction and spread footing type foundations are a feasible alternative for this project.

Mat foundations are generally advantageous when it is necessary to transmit relatively heavy or widely distributed loads onto a weak soil profile. However, for many of the same reasons stated for the spread footing alternative above with regard to the soil conditions, we do not believe a conventional mat foundation is feasible for this project. We also considered use of a fully- or partially- compensated mat foundation for this site.

In a fully-compensated mat foundation, a volume of soil equal in weight to the proposed structure and mat foundation is excavated. The structural mat foundation is then constructed on the bottom of the excavation followed by construction of the rest of the building. The end result is that no net increase in the stress acting on the soil beneath the mat foundation occurs. The concept of a partially-compensated mat foundation is similar, with the exception that only a portion of the weight of the structure is offset by the weight of excavated soil, resulting in a proportionate net increase in stress acting on the soil after the structure has been constructed. Thus, it can be seen that by increasing the depth of excavation, the pressure that can safely be exerted by a structure is correspondingly increased. In some instances, this may require construction of a vacant basement area beneath the structure to accomplish the desired result.

Considering the potential environmentally sensitive nature of the soils existing at this site, however, the relative cost of handling and/or disposing of such an amount of material as would be generated by constructing either a fully- or partially- compensated mat foundation may be prohibitive for this project when other foundation alternatives are considered. The costs of construction dewatering would be an added burden, making other foundation alternatives more feasible for this site.

Drilled shafts or caissons are commonly used where relatively soft or otherwise unsuitable surficial soils overlie a much stiffer or denser bearing material. Such conditions are found at this site, but at a considerable depth (approximately 80 to 82 feet) below the surface. Furthermore, the presence of a relatively high groundwater table will make construction of a drilled shaft foundation system considerably more difficult and expensive. Based on the anticipated relatively light structural loads, we do not believe drilled shafts are economically viable considering the potential for special construction considerations, i.e. disposal of significant quantities of environmentally impacted drilled soils cuttings and the possibility that deep cased shafts may be necessary to provide stability and groundwater control in the drilled pier excavations.

Various types of pile foundations may be economically used to support structures when the loads and/or soil conditions require the foundation to be supported at a considerable depth below the surface. Such conditions exist for this site and, in our judgment, an end-bearing pile foundation system supported in the hard sandy clay layer at depths of about 70 feet below the surface in Boring B4 or on sound shale bedrock at depths of approximately 80 to 82 feet below the existing ground surface would be the best suited type of foundation system for the proposed development. At this time, we do not anticipate shallower friction type bearing piles will be cost effective for this site, given the potentially significant reduction in available pile capacity due to downdrag, or negative skin friction, which may occur as a result of consolidation of the surficial fill and underlying, normally consolidated soils.

Various types of deep pile foundations, including top-driven steel pipe or H-pile sections, prestressed concrete piles, and auger cast piles are available. In our experience, auger cast piles are generally more economical than driven steel or prestressed concrete piles. Furthermore, the undesirable effects of noise and vibration associated with conventional pile driving operations can be avoided by using auger cast piles. It should be noted, however, no direct method of inspection of the bored hole, the bearing stratum or the integrity of the cast-in-place concrete may be possible for the auger cast piles. Therefore, we believe top driven steel piles (pipe or H-sections) may be better suited for this site provided the noise and vibrations associated with driving piles into competent bearing materials generally encountered below depths of about 70 feet (approximate relative elevation 30) below the existing ground surface in the vicinity of Boring B4 are tolerable. However, steel piles are subject to corrosion and the corrosion may be severe in some fills. Since the fill materials encountered at this site are miscellaneous in nature, proper protection of the piles such as concrete encasement or coatings should be provided in the most vulnerable zones. As the top driven steel piles and auger cast piles each have their own advantages, we have included preliminary design recommendations for both types of piles in the following "Recommended Feasible Foundation Systems" section of this report.

The GeoPier ™ soil reinforcement method provides a design-build alternative to deep foundations or over excavation and replacement of unsuitable materials with engineered fill by using a Rammed Aggregate Pier™ system. In using this system, very stiff and highly densified short aggregate pier elements are installed in cavities made within the existing fill and/or very soft and highly organic soil layers. Although the existing fill and organic soils are penetrated by the pier cavities, the pier bottoms often terminate on soft and compressible, underlying inorganic soils. Generally, the GeoPier elements are installed in a grid pattern, typically from 8 to 15 foot centers.

The GeoPier ™ system is a proprietary technology introduced by GEOPIER™ Foundation Company, Inc. (GFCI). It is claimed by GFCI that the GeoPier ™ soil reinforcement has been used on over 300 projects nationwide, and has been used on many projects for the express purpose of improving subgrade soils for eventual floor slab support. Additionally, this system boasts of substantial cost savings compared with over-excavation and replacement methods and deep foundation systems. If you are interested in pursuing this alternative further, we would be pleased to provided you with contact information for GFCI.

2. Recommended Feasible Foundation Systems

a. Top Driven Steel H-Piles or Steel Pipe Piles - As the subsurface conditions call for considerable pile lengths and potentially hard driving in the final set, we recommend use of either concrete filled steel pipe piles or steel H-piles if this foundation alternative is chosen. Steel H-piles penetrate into the ground more readily than pipe piles because they displace less material. However, driven steel pipe piles may be less costly per foot than steel H-piles and construction inspection is easier for pipe piles as damaged and/or deflected H-piles may go undetected during driving through hard soils.

The proposed buildings at this site may be supported on top-driven steel H-pile sections driven a few feet into competent shale anticipated to be present at depths of about 80 to 85 feet (approximate relative elevation 20) below the surface of the site. Our experience coupled with static calculations based on point bearing capacity components of the shale bearing stratum encountered at this site indicate steel H-piles with a minimum 12 inch flange width are capable of developing individual working capacities on the order of 60 to 80 tons. This recommended allowable working capacity is based on using a minimum factor of safety of 2.5 from the calculated ultimate value. In order to develop the recommended design capacity and required depth of embedment within the shale stratum, we anticipate a relatively heavy pile section (HP 12 X 53 as a minimum) will be necessary to resist the driving stresses. Somewhat lower capacity H-piles requiring a lighter section and terminating in the hard sandy clay layer encountered at a depth of about 68½ feet (approximate relative elevation 32½) below the surface in Boring B4 may also be feasible for this project.

Closed-end concrete filled pipe piles are also feasible for this project. However, due to their higher displacement, we anticipate they will be more difficult to drive and seat than H-section piles. Where nominal 10 inch diameter pipe piles with a standard wall thickness of 0.365 inches (Schedule 40) terminating in the hard sandy clay layer encountered at a depth of about 68½ feet (approximate relative elevation 32½) below the surface in Boring B4 are used, individual working capacities of approximately 10 to 20 tons per pile is recommended for design. Where similar pipe piles are driven a foot or two into competent shale encountered at a relative elevation of about 20 in Boring B4, design working capacities on the order of 30 to 40 tons per pile could be used for design. Use of these values assumes a minimum factor of safety of about 2.5. Somewhat higher capacity pipe piles are feasible for this project using a larger diameter, heavier walled pipe driven to somewhat lower tip elevations in the competent shale. Mandrel driven pipe piles could achieve considerably higher design capacities.

The design working capacities recommended above for the steel H-piles and pipe piles assume any allowable skin friction available to depths of about 60 feet below the existing ground surface is roughly equal to the ultimate negative skin friction (downdrag) that may occur due to consolidation of the existing over burden fill and underlying normally consolidated, organic soils. If increases in the effective vertical stress within the overburden soil deposits are anticipated after the piles are installed (such as those resulting from increased surficial surcharge loads or a lowering of the groundwater table), an appropriate reduction in the allowable pile capacity should made. After the preliminary foundation design is completed, we would be pleased to assist with this evaluation. The piles should also be evaluated for reduction of allowable working load capacity due to group action. After the preliminary foundation design is completed, we would be pleased to assist with this evaluation.

Regardless of the type of piling used, we recommend at least one vertical compression load test be performed prior to production pile installation to verify suitability of the bearing stratum for the design pile working capacity. Additionally, we strongly recommend the contractor who is retained to perform the pile driving operations be required by the project specifications to submit a wave equation analysis for the proposed pile, hammer and cushion system. The wave equation analysis will provide the final set criteria for the design pile capacity while demonstrating the tip stresses developed during the driving operations will not damage the piles. If the wave equation analysis or field experience suggest the driving stresses in the piles may become excessive, it may be necessary to reinforce the pile points with a driving shoe, pre-drill pilot holes for the piles, or use a heavier pile section. If pre-drilling is required for the H-pile alternative, it may be necessary to grout the pilot holes to provide the necessary skin friction component of the total allowable pile capacity.

Any piles which are damaged during the driving operations should be replaced with another competent pile. Piles which encounter refusal at elevations significantly above the expected depth of the planned bearing stratum should be presumed to have stopped on an obstruction and be rejected as having an indeterminate capacity. Any grade beams or pile caps associated with the proposed foundation system should have their bottoms embedded a minimum of 42 inches below final exterior site grade to provide protection against the effects of frost heave during normal winters.

With regard to settlement, we estimate total post-construction settlement for either the steel H-pile or pipe pile foundation alternative, where they are designed and constructed in accordance with the recommendations presented above, will be on the order of ½ inch or less. This estimate is based on the anticipated loading conditions and our past experience with similar soils and foundation systems.

<u>b. Auger Cast Piles</u> - Considering the potentially detrimental noise and vibration concerns associated with pile driving operations and the necessity to protect steel piles from corrosion within the fill deposits, an auger cast pile system may be advantageous for support of the proposed condominium buildings at this site.

For nominal 14 to 18 inch diameter auger cast piles, we anticipate allowable working capacities ranging from 30 to 50 tons per pile be used for design purposes where they are extended a foot or so into the competent shale stratum encountered in our Boring B4 at approximately relative elevation 20. Again, use of this recommended design pile capacity is based on a minimum factor of safety of 2.5 from the calculated ultimate value. The recommended allowable pile capacity also assumes any allowable skin friction available above a depth of about 60 feet is about equal to the ultimate negative skin friction (downdrag) that may occur due to consolidation of the existing over burden fill and underlying normally consolidated, organic soils. The piles should also be evaluated for reduction of allowable working load capacity due to group action. After the preliminary foundation design is completed, we would be pleased to assist with these evaluations if desired.

Piles which encounter auger refusal at elevations higher than the recommended design elevation should be presumed to have stopped on an obstruction and be rejected as having an indeterminate capacity. To provide protection against uplift due to frost heave during normal winters, pile caps and/or grade beams should be embedded a minimum of 42 inches below final site grade.

The auger-cast piles should be installed by an experienced contractor using appropriate equipment to assure the piles are cast in intimate contact with the soil and/or bedrock, free of any discontinuities or soil intrusions. In this regard, the piles should be placed by rotating a continuous flight, hollow-shaft auger with no gaps or other breaks into the ground to the design depth. High-strength grout should then be pumped with sufficient pressure in tremie fashion through the hollow auger shaft as the auger string is withdrawn from the ground to fill the augured hole while preventing its collapse and to cause lateral penetration of the grout into the porous zones of the surrounding soil. A positive head of several feet of grout should be maintained on the perimeter auger flights above the injection point at all times during removal of the augers to ensure the grout completely displaces all loose soil and groundwater from the hole.

If any reinforcement in the pile is required, it must be placed before the grout takes its initial set. After a pile has been completed, adjacent piles should not be installed until the grout has reached its initial set to minimize the possibility of interconnection between adjacent piles while the grout is still in a fluid state. In this regard, we recommend new piles not be installed within six pile diameters (center-to-center) of an existing pile until the concrete is at least 24 hours old. A minimum center-to-center pile spacing of twice the nominal diameter of the piles is recommended.

The high-strength grout should consist of a mixture of Portland cement, sand which meets the requirements of ASTM Standard C-33, and potable water. Admixtures such as pozzalons, mineral fillers and/or plasticizers may be added at the designer’s discretion. The materials should be proportioned to produce a homogenous grout capable of achieving the minimum specified 28-day compressive strength when tested in accordance with ASTM C-109 (Compressive Strength of Hydraulic Cement Mortars [Using 2-inch Cube Specimens]). The consistency of the grout should be such that it can be readily placed but is also capable of maintaining the solids in suspension without significant water gain. The piling contractor should be required by the project specifications to submit a mix design for the high-strength grout for approval prior to commencing the work.

As with the driven piles, we recommend at least one vertical compression load test be performed prior to production pile installation to verify suitability of the bearing stratum for the design pile working capacity. We also strongly recommend the installation of the auger cast piles be monitored by a representative of the soils engineer to verify proper installation procedures and materials are used and the design pile lengths are achieved.

With regard to settlement, we estimate total post-construction settlement for the auger cast pile foundation alternative, where the piles are designed and constructed in accordance with the recommendations presented herein, will be on the order of ½ inch or less. This estimate is based on the anticipated loading conditions and our past experience with similar soils.

B. Grade Slab Considerations

Demolition to remove any existing buildings and/or below-grade structures within the proposed building areas should include complete removal of any existing foundations and slabs. Any existing utilities located within the proposed building areas should be properly abandoned or be removed and rerouted around the proposed building areas prior to construction. As a minimum, proper abandonment of underground utilities should involve complete removal and legal disposal of any product, residue, soil, etc. and pressure grouting the complete void space within the utility line. Excavations resulting from the removal of existing structures and underground utilities should be backfilled with engineered fill, placed and compacted in accordance with the recommendations outlined later in this report.

As previously noted, the soil profile at this site contains approximately 6 to 19 feet of surficial miscellaneous fill with variable amounts of organics and wood overlying soft/loose compressible organic “river bottom” type soils which extend to depths of over 30 feet below the surface in some of the soil borings. The anticipated floor loadings and any new surcharge loadings associated with raising the existing ground surface with additional fill material will likely increase the effective vertical soil stress beneath the existing fill deposit. This will further aggravate the potential for additional consolidation settlement of the underlying natural soil layers to occur. For these reasons, we do not believe the existing miscellaneous fill and underlying organic soils are suitable for support of the proposed floor slabs-on-grade due to the risks of excessive total and differential settlement and premature distress, including cracking and/or faulting of the floor slabs.

Given the thickness and amount of the unsuitable fill deposits and the underlying organic, normally consolidated alluvial soils encountered at this site, and considering their apparent environmentally sensitive nature and the relative position of the apparent hydrostatic groundwater level at this site, mass excavation of the existing unsuitable soils and replacement with engineered fill will likely be cost prohibitive for the proposed building slab-on-grade areas.

We have also considered implementation of various soil improvement/reinforcement methods and/or ground modification techniques, such as surcharging the site, use of geotextiles, dynamic deep compaction, vibratory replacement, stone columns and chemical injection grouting. While, any of these methods may help to mitigate the risk of potential settlement, cracking and/or faulting of the floor slabs to varying degrees, most at a substantial cost, they will not eliminate the risk entirely.

Therefore, in lieu of implementing the less reliable soil improvement or ground modification options available for the proposed building grade slabs, and to eliminate the risk of excessive grade slab settlement, cracking and/or faulting, we recommend use of structurally supported floors for the proposed condominium buildings. These structural slabs could be supported on a series of grade beams which, in turn, are supported on the previously recommended deep foundation system for the proposed buildings at this site.

It should be noted, we have previously recommended consideration of the GeoPier ™ Rammed Aggregate Pier™ soil reinforcement method which may offer acceptable alternative solutions for grade slab construction at this site. However, design, installation and warranty of the GeoPier ™ method is proprietary and beyond the scope of this report.

<u>C. Pavement Subgrade Preparation</u>

For the same reasons stated in the preceding section, the existing miscellaneous fill deposits and underlying variably organic "river bottom" type soil profile are judged to be unsuitable for support of pavements, sidewalks and similar types of site improvements. Realizing somewhat variable settlements and/or cracking of the pavements may be tolerable, however, we anticipate very limited modifications to the existing pavement subgrade soils will be made with the expectation that reduced design life and increased annual maintenance will occur. We cannot readily quantify the amount of settlement or severity of distress likely to occur due to the variable nature of the existing fill deposit and random occurrence of organic materials within the underlying natural soils, but we believe at least a couple inches of differential settlement is possible during the life of the facility. If this magnitude of settlement is acceptable, we offer the following comments and preliminary recommendations for pavement and site work subgrade preparation.

Soils having an organic content greater than 5 percent are generally considered unsuitable for support of structures due to the risk of excessive long term settlement. Therefore, we recommend all existing surficial topsoil fill and surface vegetation be completely removed from areas of proposed pavements, walkways, etc. and be wasted or stockpiled for later landscaping use. Based on our soil boring information, surficial topsoil fill materials are expected to range in thickness from a few inches to about 1 foot across much of the proposed site. It should be anticipated that somewhat thicker localized "pockets" of organic topsoil fill will be encountered during construction which will require more extensive stripping. For this reason, we recommend the stripping and earthwork operations be closely monitored by qualified personnel in the field at the time of construction to verify the existing topsoil and surface vegetation are sufficiently removed prior to construction.

Given the past development of the proposed site, some demolition of existing structures may be necessary prior to construction of the proposed development. In this regard, we recommend all existing concrete foundations, slabs and similar types of at- or below- grade structures be completely removed from areas of proposed new pavements to a depth of at least 30 inches below the planned final pavement subgrade elevation to minimize the possibility of forming stress concentration points beneath the new pavements. Furthermore, we recommend the root mats of any trees or tree stumps be completely grubbed out of areas of proposed development and the resulting excavations be backfilled with engineered fill. Excavations resulting from the removal of existing structures, underground utilities and tree stumps should be backfilled with engineered fill, placed and compacted in accordance with the recommendations outlined in the following section of this report.

Following the selective demolition, topsoil stripping and rough grading activities noted above, we anticipate the exposed subgrade soils in the proposed parking lot areas will generally consist of the existing deposits of variable to miscellaneous fill materials encountered in our soil borings. We recommend these exposed subgrade soils be thoroughly proofrolled and/or compacted as described below to achieve a higher, more uniform density. In this regard, we recommend exposed cohesive (clayey) subgrade soils be thoroughly proofrolled under the observation of a qualified soils engineer using a fully loaded, tandem axle dump truck or other heavily loaded pneumatic-tired vehicle. The proofrolling should be performed by making continuous side-by-side passes across the entire subgrade area within the proposed pavement limits. Where the exposed subgrade materials consist of existing granular fill materials, we recommend they be scarified, moisture conditioned as necessary and compacted until at least the top 12 inches of existing material is compacted to a minimum of 95 percent of the maximum dry density determined by ASTM Standard D-1557 (Modified proctor) or until no further increase in density can be readily achieved. These operations will help to uniformly compact the subgrade surface and locate any loose or soft areas which may require stabilization. Subgrade areas which yield excessively or "pump" during the proofrolling operations should be mechanically stabilized or be excavated and replaced with approved engineered fill.

Given the variable to miscellaneous nature of the anticipated pavement subgrade soils, we recommend the design pavement section incorporate use of a woven geotextile fabric between the finished subgrade and the aggregate base course layer to provided extra subgrade reinforcement. Mirafi 500X or equivalent, placed in strict accordance with the manufacturer’s recommendations, would be suitable for this purpose.

We also recommend close observation and testing of the subgrade soils be performed during the earthwork and foundation construction operations. Any highly miscellaneous and/or deleterious materials, such as wood or debris, encountered during the construction activities at this site should be removed from areas of proposed pavements and site work and be replaced with engineered fill. The judgment to remove miscellaneous or deleterious materials should be made in the field when the excavations for utilities, pile caps and grade beams are opened and the extent of such conditions will be more apparent.

If it becomes necessary to remove unstable or otherwise unsuitable existing soils during the proofrolling and/or compaction operations recommended above, the following corrective guidelines are recommended. Where the unstable subgrade consists of soft or wet cohesive soil or highly organic or otherwise deleterious material, we recommend the unstable soil be undercut as needed up to a maximum depth of 3 feet below the planned final subgrade elevation. If the soil at this depth is still unstable or otherwise unsuitable, a biaxially oriented geogrid, such as Tensar BX geogrids or equivalent, should be placed over the exposed excavation bottom to provide increased shearing resistance and reinforcement. The geogrid should extend laterally beyond the perimeter of the unstable soil area and be overlapped along adjacent roll widths in accordance with the manufacturer's recommendations. The excavation should then be backfilled to the planned final pavement subgrade elevation depth using engineered fill material, placed and compacted in accordance with the recommendations outlined in the following section of this report. Due to the wide variety of potential subgrade stability problems, the field operations should be monitored and documented by an experienced geotechnical engineer capable of making appropriate decisions in the field at the time these observations are made.

As previously noted, the pavement subgrade materials are generally expected to consist of the variable to miscellaneous, very loose to medium dense granular fill materials (predominant) or stiff sandy clay fill materials (isolated) encountered in our soil borings. Considering the previously mentioned risks associated with using these materials for pavement support, we judge them to have poor subgrade support characteristics. On this basis, we recommend use of a subgrade resilient modulus (M_R) of 3,000 pounds per square inch (psi) for flexible pavement design and a modulus of subgrade reaction (k) of 100 pounds per cubic inch for design of concrete pavements. Use of these values assumes the pavement subgrade preparation recommendations, as outlined previously in this report, will be implemented prior to constructing any new pavements at the site.

To help mitigate possible premature pavement distress, consideration might be given to delaying placement of the wearing course layer to allow time for some of the settlement/distress to occur. Wherever possible, the use of lightweight fill materials should be used when raising the existing site grades to the planned final subgrade elevation. Lightweight fill materials must be of a sound, durable nature and not be prone to expansion or cyclical freeze-thaw degradation. Consideration should also be given to minimizing use of grade supported site improvements to the extent practical. In this regard, it may prudent to use berms as opposed to retaining walls and treated wood decking in lieu of concrete walks. Similarly, swimming pools, tennis courts, and similar types of amenities should be located on structurally supported floors or deep foundations to avoid future problems associated with long term settlement.

D. Engineered Fill Requirements

All engineered fill for this project, including any utility trench and foundation excavation backfill, should be an approved material free of frozen chunks, organics, debris or other deleterious material. The fill should be spread in level layers not exceeding 10 inches in loose thickness, with each layer being compacted to a least 95 percent of the maximum dry density determined by ASTM Standard D-1557 (Modified Proctor). A sufficient number of field density tests should be performed during placement to verify proper compaction is achieved. Fill material should never be placed on frozen or muddy ground.

In general, existing site soils may be reused as engineered fill, provided all organic or otherwise deleterious materials are completely removed from the proposed fill material before it is placed and compacted. To facilitate compaction, we recommend any granular fill materials be placed within ±4 percent of the optimum moisture content determined by ASTM Standard D-1557 (Modified Proctor). If necessary to achieve this condition, appropriate moisture reconditioning should be performed at the time of placement. If it is necessary to add moisture, we recommend it be done by disking and harrowing the soil as the water is added to provide a relatively uniform moisture content throughout the soil mass. Alternately, if the soil is too wet at the time of placement, we recommend it be disked and aerated to allow it to dry to the desired moisture content prior to compaction, weather conditions permitting.

Cohesive soils may also be used or re-used for engineered fill, however, cohesive soils are more sensitive to the moisture content at which they are placed to achieve proper compaction. Therefore, if any cohesive fill soils are used, we recommend they be placed at a moisture content within ±2 percent of the optimum moisture content determined by the Modified Proctor test. Moisture reconditioning, if needed, could be accomplished using the same methods described above for granular soil, however, it should be anticipated considerably more time and effort will likely be required to properly moisture recondition cohesive soils in comparison to granular soil using this technique. Cohesive fill materials should be compacted using a heavy sheepsfoot roller to knead the soils together and help ensure all larger chunks are broken down into an integrated soil mass free of large air voids.

E. Swimming Pool Design Considerations

As mentioned earlier in this report, the apparent hydrostatic groundwater level at this site appeared to be between depths of about 6 to 16 feet (approximate relative elevations 94 to 88) below the existing ground surface at the times our soil borings were performed. The final swimming pool design details are not available at this time. However, based on our understanding of the proposed development, the proposed swimming pool will have a maximum depth of about 6 feet at its deepest point below the proposed ground floor finished grade elevation, which is assumed to be about 102 at this time. On this basis, it appears the bottom of the proposed swimming pool will slightly higher than the prevailing groundwater level at this site. However, considering the level of Lake Huron has been several feet lower than its historical average in recent months, a permanent subsurface drainage system may be prudent to minimize hydrostatic uplift on the bottom of the pool at times when the pool may be emptied of water.

To help further minimize the potential for excessive hydrostatic uplift pressures to develop on the base of the swimming pool if the pool is emptied, we recommend a fail-safe method of monitoring the water level in the backfill soils around the pool be provided. In this regard, several piezometers could be installed within the granular backfill soils surrounding the pool. These piezometers should be observed before any lowering of the water level in the pool to verify elevated or perched groundwater levels have not developed in the backfill soils.

Considering the close proximity of the Saginaw River and its potential to fluctuate by several feet or more on a seasonal basis, we recommend appropriate design considerations be provided to prevent or resist any potential hydrostatic uplift forces acting on the bottom of the pool when it is emptied and to counteract normal high water level fluctuations. This could be accomplished in a number of different ways, including the following:

- Provide sufficient weight and rigidity in the pool bottom and walls to resist the maximum anticipated hydrostatic uplift pressures.
- Provide relief valves in the bottom of the pool to relieve the maximum anticipated hydrostatic uplift pressures. The valves should be properly filtered to prevent fine-grained soils from entering the pool should the valves be activated.
- Install a permanent, properly filtered subsurface drainage system connected to a continuous-acting pump which could be engaged sufficiently in advance of any planned draining of the pool to prevent hydrostatic uplift from occurring.

F. Recommended Additional Subsurface Investigation

Since only one soil boring was extended to competent material for support of end-bearing piles, we recommend additional subsurface investigation and analysis be performed at this site prior to finalizing any deep foundation design plans. The strength and integrity of the shale bedrock should be determined as part of this additional subsurface investigation. As a minimum, we recommend two additional soil borings and rock cores be performed to verify the elevation and character of the recommended bearing stratum and finalize design recommendations for pile foundations.

VI. PRELIMINARY CONSTRUCTION CONSIDERATIONS

We do not anticipate any serious geotechnical problems associated with constructing steel pipe pile, steel H-pile or auger-cast pile foundation systems as described herein. Considering the somewhat variable character of the existing fill deposit encountered at this site, some problems with drilling/driving through the existing fill deposits should be anticipated. Although not apparent in our soil borings, it is possible some large pieces of debris or existing buried concrete structures may obstruct the augering/driving operations making it necessary to pre-excavate through the fill deposit to install or drive the auger cast piles or steel piles, respectively, in some areas of the site. Due to the hard to very hard nature of the anticipated pile bearing stratum, some difficulty in augering/driving through this material should also be anticipated. Irrespective of the type of pile foundation system used, it is imperative the installation work be performed by an experienced, qualified contractor familiar with the local subsurface conditions.

Considering the apparent position of the long term hydrostatic groundwater level relative to the anticipated depth of excavation for pile caps and/or grade beams, we do not believe groundwater seepage will be a serious concern during construction. Depending on the time of the year construction operations take place, it is possible perched groundwater accumulations may be encountered in the somewhat miscellaneous fill materials encountered in the upper soil profile. Excavations for pile caps and grade beams should be maintained in a dry condition until they are completely backfilled. Therefore, it should be anticipated that some pumping from construction sumps will be needed to remove water accumulations resulting from perched groundwater seepage, precipitation or surface runoff. We believe these water accumulations will be manageable using standard sump pit and filtered pumping procedures.

During construction of the proposed swimming pool or should deeper excavations be required at the site, such as for utility cuts or to remove and replace existing unsuitable fill materials with engineered fill, some minor geotechnical problems may be encountered with regard to management of groundwater and maintaining stability of excavation sidewalls. Where excavations extend only a foot or two below the prevailing groundwater level, estimated to be at depths of approximately 6 to 18 feet below the existing ground surface, we anticipate groundwater accumulations will be manageable using standard sump pit and filtered pumping procedures. Where excavations extend more than a foot or so below the prevailing groundwater level, it may be necessary to implement more substantial dewatering techniques such as pumping from slotted casings or well points installed several feet below the anticipated excavation bottom. To better gage the rate and volume of groundwater seepage likely to be encountered during construction, consideration should be given to performing a number of test pits and/or pump tests prior to the start of any excavation work. This will help enable the excavator to determine the extent of the dewatering system required for timely and productive excavation work.

Some sloughing or caving of the existing surficial fill soils should also be expected to occur during construction, particularly in conjunction with any groundwater seepage that may occur. For this reason, we recommend pile caps and grade beams be formed to maintain vertical sides and minimize problems associated with caving soils. All excavations must be properly braced or sloped to comply with MI-OSHA regulations and to provide a safe work place for construction personnel. The practice of stockpiling excavated soils adjacent to excavations is not recommended as this surcharge loading may cause sudden collapse of the excavation sidewalls. If material and/or equipment is to be stored or operated near an excavation, additional bracing or shoring must be provided to resist these heavier surcharge loadings.

After the site has been stripped to the planned subgrade elevations, the exposed subgrade is expected to consist primarily of the existing, somewhat variable granular and cohesive fill deposits. These materials are somewhat prone to disturbance by the anticipated construction operations, especially in the presence of moisture. Therefore, exposed subgrade areas should always be properly graded during construction to provide positive surface drainage away from exposed excavations and subgrade areas and to prevent precipitation and surface runoff from ponding on the site. If problems with excessive pumping and/or rutting of subgrade soils persist during construction, it may be necessary to disc and aerate them. If inclement weather hinders attempts to properly aerate the subgrade soils, it may become more economical to remove and replace them with a layer of well-graded, clean (less than 6 percent passing a No. 200 sieve), crushed aggregate to provide a stable surface upon which to place engineered fill and structures.

Depending on the type of piling used, it would be prudent to check each pile for heave due to displacement of cohesive soils as subsequent piles are driven. This is especially important for closed-end pipe piling. In this regard, we recommend the top end of each pile be verified immediately after driving (and before any subsequent pile is driven) using a distant benchmark that will not be affected by the pile driving operations. After all the piles have been installed, the top elevation of each pile should be rechecked to determine if any heaving has occurred. Any piles which are determined to have heaved more than 0.2 inches should be reseated to their original position. We also recommend a crack condition and elevation survey of all surrounding structures be taken prior to any pile driving operations to verify the extent to which the pile driving operations may affect these existing structures.

VII. GENERAL COMMENTS

Samples taken in the field will be retained in our laboratory for a period of sixty days from the date of this report and will then be disposed of, unless otherwise requested. Samples stored over an extended period of time are subject to moisture loss and are no longer representative of the in-situ condition in which they were sampled.

During the course of this soil evaluation, procedures were followed which represent accepted practice in the field of geotechnical engineering. Therefore, discrepancies may exist between the driller's field logs and the final Soil Boring Logs submitted with this report. Field logs are prepared during the drilling and sampling operations and describe field occurrences, sampling locations and other relevant information. The engineer preparing the report reviews the field logs as well as the laboratory soil classifications and the laboratory test data. The final Soil Boring Logs are then promulgated based on all the information available from the field and laboratory operations.

Considering the potential extent of construction concerns for this project, we recommend the contract specifications include the following or its equivalent: "The contractor will, upon becoming aware of subsurface or latent physical conditions differing from those disclosed by the original soil investigation work, promptly notify the owner verbally to permit verification of the conditions, and in writing, as to the nature and extent of the differing conditions. No claims by the contractor for any conditions differing from those anticipated in the plans and specifications and disclosed by the soil studies will be allowed unless the contractor has so notified the owner, verbally and in writing, as required above, of such changed conditions."

The services of WAK should be engaged during construction to monitor the earthwork and foundation activities and to verify proper materials, equipment and procedures are used. An appropriate number of field density tests should be performed during the earthwork operations to verify proper compaction is achieved where engineered fill is used. Likewise, the installation of pile foundations should be closely monitored to verify conditions are similar to those anticipated at the time our recommendations were formulated and to verify adequate pile penetration, set and/or alignment are achieved.

REPORT PREPARED BY:

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REPORT REVIEWED BY:

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Summary of Soil Data

Proposed Marina Place Condominiums
Bay City, Michigan
WAK Project No. 00-0881-0626
(Page 2 of 2)

Boring No.	Sample No.	Sample Depth (Feet)	Blow Count	N (bfp)	Qp (tsf)	c (tsf)	w (%)	USCS Group Symbol
4	1	1 to 2-1/2	3 - 3 - 4	7	---	---	20	OL-MISC. FILL
	2	3-1/2 to 5	3 - 4 - 5	9	1-1/2	---	29	CL-OL-MISC. FILL
	3	6 to 7-1/2	1 - 1 - 3	4	---	---	---	SP-OL-MISC. FILL
	4	8-1/2 to 10	1 - 1 - 2	3	---	---	28	SP-OL-MISC. FILL
	5	13-1/2 to 15	5 - 2 - 2	4	DS	DS	171	SP-OL-MISC. FILL
	6	18-1/2 to 20	2 - 1 - 1	2	DS	DS	172	ML-OL
	7	23-1/2 to 25	1 - 1 - 1	2	<1/4	0.1	76	ML-OL
	8	25 to 27	Shelby Tube	---	NR	NR	NR	ML-OL
	9	28-1/2 to 30	WH - WH - 1	1	<1/4	0.1	34	ML-OL
	10	33-1/2 to 35	WH - WH - 1	1	<1/4	0.1	63	CL
	11	38-1/2 to 40	WH - WH - 1	1	<1/4	0.1	60	CL
	12	41 to 43	Shelby Tube	---	NR	NR	NR	CL to SP
	13	43-1/2 to 45	5 - 8 - 7	15	---	---	23	SP
	14	48-1/2 to 50	9 - 13 - 7	20	---	---	20	SP
	15	53-1/2 to 55	8 - 6 - 8	14	---	---	23	SP
	16	58-1/2 to 60	5 - 6 - 5	11	1	0.5	27	CL
	17	63 to 65	Shelby Tube	---	1-3/4	---	23	CL
	18	68-1/2 to 70	16 - 20 - 22	42	>4-1/'2	---	10	CL-ML
	19	73-1/2 to 75	8 - 16 - 22	38	>4-1/'2	---	10	CL-ML
	20	78-1/2 to 80	26 - 50/1"	50/1"	>4-1/'2	---	15	CL-SHALE
	21	83-1/2 to 85	100/4"	100/4"	>4-1/'2	---	13	SHALE
5	1	1 to 2-1/2	2 - 2 - 2	4	---	---	47	SP-MISC. FILL
	2	3-1/2 to 5	6 - 11 - 9	20	---	---	6	SP-MISC. FILL
	3	6 to 7-1/2	2 - 1 - 1	2	---	---	79	SP-MISC. FILL
	4	8-1/2 to 10	2 - 2 - 3	5	---	---	20	SP-MISC. FILL to SM-OL

WH = Weight of Hammer for 6 Inches of Penetration
NR = No Recovery

APPENDIX

General Notes
Soil Boring Location Diagram
Soil Boring Logs (B1 through B5)
Summary of Soil Data (Pages 1 and 2)
Unified Soil Classification System

SOIL BORING LOG

Project: PROPOSED MARINA PLACE CONDOMINIUMS

Location: BAY CITY, MI

Boring No: B1 (1 OF 2)

WAK Job No: 00-0881-0626

Client: MARINA PLACE L.L.C.

Description of Material Ground surface elevation: ~99	DEPTH (ft)	Sample Length	Sample I.D.	Sample Type	Standard Penetration 'N' Blows Per Ft.	Hand Penetrometer qp (tsf)	Unconfined compression qu (tsf)	Torvane Shear cs (tsf)	Moisture Content w (%)	Natural Unit Weight γ (pcf)
Driller reported 1' of sandy topsoil FILL – dark brown – moist (SM-OL-FILL)			1	SS	6				20	
Silty sand FILL; trace gravel, slag and clay, few roots – dark brown to brown – moist – loose (SM-FILL)	5		2	SS	10				17	
Fine to medium SAND; trace silt, few silty clay seams – gray – moist – medium dense (SP)			3	SS	5	$1\frac{1}{2}$		0.8	14	
Silty CLAY; some sand, trace gravel, few sand lenses – gray – moist – stiff (CL)										
Clayey SILT; some sand, trace to some organics, trace vegetation, few sand lenses and shells, few silty peat pockets – brownish gray, becoming gray below 13' – very moist – firm (ML-OL)	10		4	SS	2	$\frac{1}{2}$		0.3	36	
	15		5	SS	2	$\frac{1}{2}$		0.3	34	
($18\frac{1}{2}$'-33') Clayey SILT; some sand, trace organics, occasional sand lenses and silt partings, few shells – gray to dark gray – very moist – firm (ML)	20		6	SS	3	$\frac{1}{2}$		0.2	50	
	25		7	SS	3	$\frac{1}{2}$		0.2	61	

(Cont on 2 of 2)

Summary of Soil Data

Proposed Marina Place Condominiums
Bay City, Michigan
WAK Project No. 00-0881-0626
(Page 1 of 2)

Boring No.	Sample No.	Sample Depth (Feet)	Blow Count	N (bfp)	Qp (tsf)	c (tsf)	w (%)	USCS Group Symbol
1	1	1 to 2-1/2	3 - 3 - 3	6	---	---	20	SM-FILL
	2	3-1/2 to 5	5 - 5 - 5	10	---	---	17	SP
	3	6 to 7-1/2	3 - 3 - 2	5	1-1/2	0.8	14	CL
	4	8-1/2 to 10	WH - 1 - 1	2	1/2	0.3	36	ML-OL
	5	13-1/2 to 15	WH - WH - 2	2	1/2	0.3	34	ML-OL
	6	18-1/2 to 20	WH - 1 - 2	3	1/2	0.2	50	ML
	7	23-1/2 to 25	WH - 1 - 2	3	1/2	0.2	61	ML
	8	28-1/2 to 30	WH - 1 - 2	3	3/4	0.3	41	ML
	9	33-1/2 to 35	WH - 1 - 1	2	1/2	0.2	53	CL
	10	38-1/2 to 40	WH - WH - 1	1	1/2	0.2	46	CL
2	1	1 to 2-1/2	6 - 6 - 8	14	---	---	14	SM-FILL
	2	3-1/2 to 5	2 - 3 - 3	6	1/DS	0.5/DS	16	CL-FILL
	3	6 to 7-1/2	10 - 6 - 4	10	---	---	12	SP
	4	8-1/2 to 10	1/12" - 1	1	<1/2	0.2	75	SP TO ML-OL-Pt
	5	13-1/2 to 15	WH - 1 - 1	2	<1/4	0.1	66	ML-OL-Pt
	6	18-1/2 to 20	WH - WH - 1	1	<1/4	0.1	39	ML
	7	23-1/2 to 25	WH - 1 - 1	2	<1/4	0.1	47	ML
	8	28-1/2 to 30	WH - 1 - 1	2	1/2	0.2	60	ML
	9	33-1/2 to 35	WH - WH - 1	1	1/2	0.2	42	ML TO CL
	10	38-1/2 to 40	WH - WH - 1	1	1/2	0.2	47	CL
3	1	1 to 2-1/2	1 - 2 - 2	4	---	---	11	SM-OL-MISC. FILL
	2	3-1/2 to 5	2 - 3 - 2	5	---	---	14	SM-OL-MISC. FILL
	3	6 to 7-1/2	2 - 2 - 2	4	---	---	22	SM-OL-MISC. FILL
	4	8-1/2 to 10	2 - 1 - 2	3	<1/2	0.2	17	SM-OL-MISC. FILL
	5	13-1/2 to 15	3 - 1 - 1	2	---		26	MISC. FILL
	6	18-1/2 to 20	3 - 2 - 2	4	<1/4	0.1	43	MISC. FILL to SC-SM-OL
	7	23-1/2 to 25	3 - 4 - 3	7	---	---	20	SP
	8	28-1/2 to 30	WH - 3 - 3	6	1/2	0.1	192	SM-OL
	9	33-1/2 to 35	WH - 1 - 2	3	1/2	0.2	51	ML
	10	38-1/2 to 40	WH - 1 - 1	2	1/2	0.2	56	ML
	11	43-1/2 to 45	WH - WH - 1	1	DS	DS	53	CL
	12	48-1/2 to 50	WH - 1 - 1	2	1/2	0.3	53	CL
	13	53-1/2 to 55	WH - WH - 2	2	1/2	0.3	44	CL
	14	58-1/2 to 60	WH - WH - 2	2	3/4	0.3	38	CL

WH = Weight of Hammer for 6 Inches of Penetration
NR = No Recovery

SOIL BORING LOG

Project: PROPOSED MARINA PLACE CONDOMINIUMS
Location: BAY CITY, MI

Boring No: B1 (2 OF 2)
WAK Job No: 00-0881-0626
Client: MARINA PLACE L.L.C.

Description of Material	DEPTH (ft)	Sample Length	Sample I.D.	Sample Type	Standard Penetration 'N' Blows Per Ft.	Hand Penetrometer qp (tsf)	Unconfined compression qu (tsf)	Torvane Shear cs (tsf)	Moisture Content w (%)	Natural Unit Weight γ (pcf)
($18\frac{1}{2}$'-33') Clayey SILT; some sand, trace organics, occasional sand lenses and silt partings, few shells – gray to dark gray – very moist – firm (ML)	30		8	SS	3	$\frac{3}{4}$		0.3	41	
Silty CLAY; trace to some sand – gray – very moist – firm (CL)	35		9	SS	2	$\frac{1}{2}$		0.2	53	
	40		10	SS	1	$\frac{1}{2}$		0.2	46	
Boring terminated at 40'										

NOTE: Changes in soil stratification indicated by lines are approximate. In situ, the transition between materials maybe gradual unless otherwise noted. The bored hole was backfilled with natural soil.

WATER LEVEL OBSERVATIONS

~$8\frac{1}{2}$' While Sampling or Drilling
38.0' Immediately After Completion
20.0' @ 4 HRS After Completion

BORING

Rig: RD CME-85 Foreman: R. RAU
Started: 1-10-01 Drawn By: R. MARR
Completed: 1-10-01 Approved: U. GOLLAPUDI

SHEET

2 of 2

SOIL BORING LOG

Project: PROPOSED MARINA PLACE CONDOMINIUMS

Location: BAY CITY, MI

Boring No: B2 (1 OF 2)

WAK Job No: 00-0881-0626

Client: MARINA PLACE L.L.C.

Description of Material Ground surface elevation: ~100	DEPTH (ft)	Sample Length	Sample I.D.	Sample Type	Standard Penetration 'N' Blows Per Ft.	Hand Penetrometer qp (tsf)	Unconfined compression qu (tsf)	Torvane Shear cs (tsf)	Moisture Content w (%)	Natural Unit Weight γ (pcf)
Driller reported 6" of sandy TOPSOIL FILL – dark brown – moist (SM-OL-FILL)										
Silty sand FILL; trace gravel, slag and clay, few brick pieces – dark brown – moist – medium dense (SM-FILL)			1	SS	14				14	
Silty/sandy clay FILL; trace asphalt and organics, few sand and organic pockets – gray, few black pockets – moist – stiff (CL-FILL)	5		2	SS	6	1 DS		0.5 DS	16	
Fine to medium SAND; trace coarse sand, silt and organics – brown, becoming grayish brown below about 8' – wet – medium dense to very loose (SP)			3	SS	10				12	
	10		4	SS	1	<1/2		0.2	75	
Clayey SILT; some sand, trace to some organics, few silty sand lenses and pockets, few peat layers and pockets, few shells – dark gray – very moist – soft to very soft soft (ML-OL-Pt)	15		5	SS	2	<1/4		0.1	66	
(17'-34')	20		6	SS	1	<1/4		0.1	39	
Clayey SILT; some sand, trace organics, occasional sand lenses and silt partings, few shells – gray to dark gray – very moist – very soft to firm (ML)	25		7	SS	2	<1/4		0.1	47	

(Cont on 2 of 2)

SOIL BORING LOG

Project: PROPOSED MARINA PLACE CONDOMINIUMS

Location: BAY CITY, MI

Boring No: B2 (2 OF 2)

WAK Job No: 00-0881-0626

Client: MARINA PLACE L.L.C.

Description of Material	DEPTH (ft)	Sample Length	Sample I.D.	Sample Type	Standard Penetration 'N' Blows Per Ft.	Hand Penetrometer qp (tsf)	Unconfined compression qu (tsf)	Torvane Shear cs (tsf)	Moisture Content w (%)	Natural Unit Weight γ (pcf)
(17'–34') Clayey SILT; some sand, trace organics, occasional sand lenses and silt partings, few shells – gray to dark gray – very moist – very soft to firm (ML)	30		8	SS	2	1/2		0.2	60	
Silty CLAY; trace to some sand – gray – very moist – firm (CL)	35		9	SS	1	1/2		0.2	42	
	40		10	SS	1	1/2		0.2	47	
Boring terminated at 40' NOTE: Driller reported releases of gas from the bore hole while pulling augers from the hole.										

NOTE: Changes in soil stratification indicated by lines are approximate. In situ, the transition between materials maybe gradual unless otherwise noted. The bored hole was backfilled with natural soil.

WATER LEVEL OBSERVATIONS

~6 1/2' While Sampling or Drilling

26' (RISING) Immediately After Completion

7.1' @ 1 HR After Completion

BORING

Rig: RD CME-85 Foreman: J. RAU

Started: 1-10-01 Drawn By: R. MARR

Completed: 1-10-01 Approved: U. GOLLAPUDI

SOIL BORING LOG

Project: PROPOSED MARINA PLACE CONDOMINIUMS

Location: BAY CITY, MI

Boring No: B3 (1 OF 2)

WAK Job No: 00-0881-0626

Client: MARINA PLACE L.L.C.

Description of Material Ground surface elevation: ~107	DEPTH (ft)	Sample Length	Sample I.D.	Sample Type	Standard Penetration 'N' Blows Per Ft.	Hand Penetrometer qp (tsf)	Unconfined compression qu (tsf)	Torvane Shear cs (tsf)	Moisture Content w (%)	Natural Unit Weight γ (pcf)
Driller reported 1' of sandy topsoil FILL – dark brown – moist (SM–OL–FILL)			1	SS	4				11	
Silty sand FILL; trace gravel and clay, occasional clay chunks, few concrete, wood and brick pieces – mixed, brown and dark brown – moist – loose to very loose (SM–OL–MISC. FILL)	5		2	SS	5				14	
			3	SS	4				22	
	10		4	SS	3	<1/2		0.2	17	
Mixed, silty sand and wood FILL; trace gravel, clay, organics and debris – dark brown – moist – very loose (MISC. FILL)	15		5	SS	2				26	
Silty/clayey SAND; trace to some organics, occasional silty clay seams – gray, few black pockets – very moist – loose/very soft (SC–SM–OL)	20		6	SS	4	<1/4		0.1	43	
Fine to medium SAND; trace silt, few shells – gray – wet – loose (SP)	25		7	SS	7				20	
Organic SAND and WOOD; trace to some silt – dark brown and black – very moist – loose (SM–OL)	30		8	SS	6	1/2		0.1	192	
(32'–43 1/2') Clayey SILT; some sand, trace organics, occasional sand lenses and silt partings, few shells – gray to brownish gray – very moist – firm (ML) (Cont on 2 of 2)	35		9	SS	3	1/2		0.2	51	

SOIL BORING LOG

Project: PROPOSED MARINA PLACE CONDOMINIUMS
Location: BAY CITY, MI

Boring No: B3 (2 OF 2)
WAK Job No: 00-0881-0626
Client: MARINA PLACE L.L.C.

Description of Material	DEPTH (ft)	Sample Length	Sample I.D.	Sample Type	Standard Penetration 'N' Blows Per Ft.	Hand Penetrometer qp (tsf)	Unconfined compression qu (tsf)	Torvane Shear cs (tsf)	Moisture Content w (%)	Natural Unit Weight γ (pcf)
(32'–43 1/2') Clayey SILT; some sand, trace organics, occasional sand lenses and silt partings, few shells – gray to brownish gray – very moist – firm (ML)	40	■	10	SS	2	1/2		0.2	56	
Silty CLAY; trace to some sand – gray – very moist – firm (CL)	45	■	11	SS	1	DS			53	
	50	■	12	SS	2	1/2		0.3	53	
	55	■	13	SS	2	1/2		0.3	44	
	60	■	14	SS	2	3/4		0.3	38	
Boring terminated at 60' * NOTE: The bore hole was advanced using rotary wash drilling techniques. Therefore, no groundwater levels were recorded immediately after completion of drilling.										

NOTE: Changes in soil stratification indicated by lines are approximate. In situ, the transition between materials maybe gradual unless otherwise noted. The bored hole was backfilled with natural soil.

WATER LEVEL OBSERVATIONS

~19' & ~43 1/2' While Sampling or Drilling
* Immediately After Completion
_____ @ _____ After Completion

BORING

Rig: RD-CME-85 Foreman: J. RAU
Started: 1-12-01 Drawn By: R. MARR
Completed: 1-12-01 Approved: U. GOLLAPUDI

SHEET

2 of 2

SOIL BORING LOG

Project: PROPOSED MARINA PLACE CONDOMINIUMS

Location: BAY CITY, MI

Boring No: B4 (1 OF 3)

WAK Job No: 00-0881-0626

Client: MARINA PLACE L.L.C.

Description of Material Ground surface elevation: ~101	DEPTH (ft)	Sample Length	Sample I.D.	Sample Type	Standard Penetration 'N' Blows Per Ft.	Hand Penetrometer qp (tsf)	Unconfined compression qu (tsf)	Torvane Shear cs (tsf)	Moisture Content w (%)	Natural Unit Weight γ (pcf)
Driller reported 1' of sandy topsoil FILL – dark brown – moist (SM-OL-FILL)			1	SS	7				20	
Mixed, sandy topsoil and wood FILL; some silt, few brick pieces – dark brown – moist – loose (OL-MISC. FILL)			2	SS	9	$1^1/_2$			29	
Mixed, sandy clay and wood FILL; some silt – dark brown and black – very moist – stiff (CL-OL-MISC. FILL)			3	SS	4					
Mixed, fine to medium sand and wood FILL; trace to some clay and silt – dark brown – moist to wet – loose to very loose (SP-OL-MISC. FILL)			4	SS	3				28	
			5	SS	4	DS			171	
Clayey SILT; some sand, trace to some organics, occasional sand lenses and silt partings, few shells – gray to dark gray – very moist – very soft (ML-OL)			6	SS	2	DS			172	
			7	SS	2	$<^1/_4$		0.1	76	
			8	ST		NR		NR	NR	
			9	SS	1	$<^1/_4$		0.1	34	
(33'-42') Silty CLAY; trace to some sand – gray – very moist – very soft (CL)			10	SS	1	$<^1/_4$		0.1	63	

(Cont on 2 of 3)

SOIL BORING LOG

Project: PROPOSED MARINA PLACE CONDOMINIUMS

Location: BAY CITY, MI

Boring No: B4 (2 OF 3)

WAK Job No: 00-0881-0626

Client: MARINA PLACE L.L.C.

Description of Material	DEPTH (ft)	Sample Length	Sample I.D.	Sample Type	Standard Penetration 'N' Blows Per Ft.	Hand Penetrometer qp (tsf)	Unconfined compression qu (tsf)	Torvane Shear cs (tsf)	Moisture Content w (%)	Natural Unit Weight γ (pcf)
(33'–42') Silty CLAY; trace to some sand – gray – very moist – very soft (CL)	40		11	SS	1	<1/4		0.1	60	
			12	ST		NR		NR	NR	
Fine to medium SAND; trace silt and coarse sand – gray to brownish gray – wet – medium dense (SP)	45		13	SS	15				23	
	50		14	SS	20				20	
	55		15	SS	14				23	
Silty CLAY; some sand, trace gravel, few sand lenses – gray – very moist – firm to stiff (CL)	60		16	SS	11	1		0.5	27	
Sandy CLAY; some silt, trace gravel, few sandy lenses – gray – very moist – stiff to very stiff (CL)	65		17	ST		1 3/4			23	
(68 1/2'–77 1/2') Silty CLAY; some sand, trace gravel, few sand lenses – moist – hard (CL-ML) (Cont on 3 of 3)	70		18	SS	42	4 1/2+			10	

SOIL BORING LOG

Project: PROPOSED MARINA PLACE CONDOMINIUMS
Location: BAY CITY, MI
Boring No: B4 (3 OF 3)
WAK Job No: 00-0881-0626
Client: MARINA PLACE L.L.C.

Description of Material	DEPTH (ft)	Sample Length	Sample I.D.	Sample Type	Standard Penetration 'N' Blows Per Ft.	Hand Penetrometer qp (tsf)	Unconfined compression qu (tsf)	Torvane Shear cs (tsf)	Moisture Content w (%)	Natural Unit Weight γ (pcf)
($68\frac{1}{2}$'-$77\frac{1}{2}$') Silty CLAY; some sand, trace gravel, few sand lenses - moist - hard (CL-ML)	75		19	SS	38	$4\frac{1}{2}$+			10	
Weathered clayey SHALE; some silt, occasional silty clay seams and shale fragments - dark gray to black - moist - hard (CL-SHALE)	80		20	SS	50/1"	$4\frac{1}{2}$+			15	
Clayey SHALE; some silt - gray - moist - hard (SHALE)	85		21	SS	100/4"	$4\frac{1}{2}$+			13	

Boring terminated at 85'

* Driller reported wood in sample I.D. No. 6 may have been lodged in auger bit.

** NOTE:
The bore hole was advanced using rotary wash drilling techniques. Therefore, no groundwater levels were recorded immediately after completion of drilling.

NOTE: Changes in soil stratification indicated by lines are approximate. In situ, the transition between materials maybe gradual unless otherwise noted. The bored hole was backfilled with natural soil.

WATER LEVEL OBSERVATIONS	BORING		SHEET
~$6\frac{1}{2}$' & ~42' While Sampling or Drilling	Rig: RD-CME-85	Foreman: J. RAU	3 of 3
** Immediately After Completion	Started: 1-11-01	Drawn By: R. MARR	
____ @ ____ After Completion	Completed: 1-11-01	Approved: U. GOLLAPUDI	

SOIL BORING LOG

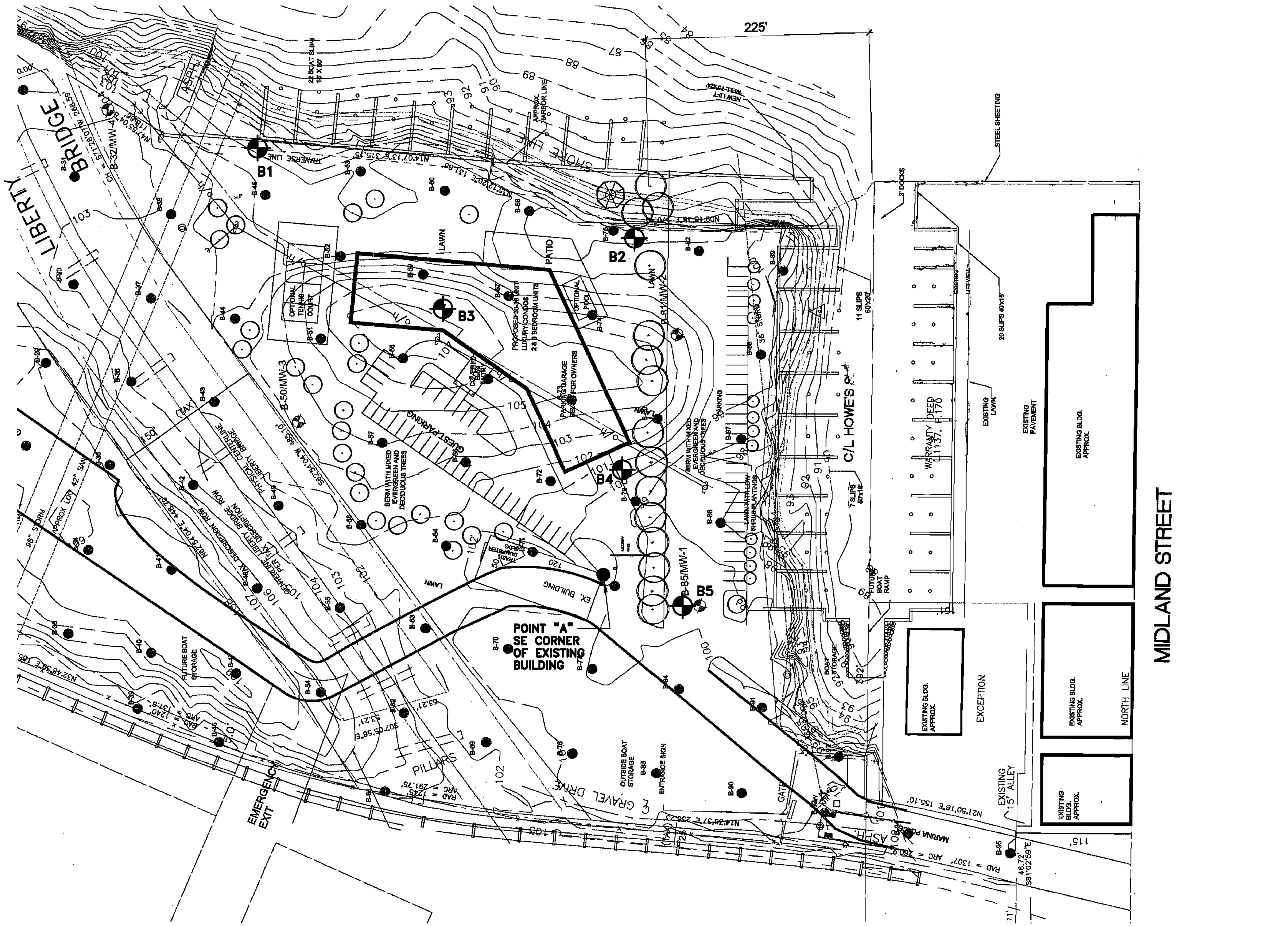
Project: PROPOSED MARINA PLACE CONDOMINIUMS
Location: BAY CITY, MI

Boring No: B5
WAK Job No: 00-0881-0626
Client: MARINA PLACE L.L.C.

Description of Material	DEPTH (ft)	Sample Length/Recovery	Sample I.D.	Sample Type	Standard Penetration 'N' Blows Per Ft.	Hand Penetrometer qp (tsf)	Unconfined compression qu (tsf)	Torvane Shear cs (tsf)	Moisture Content w (%)	Natural Unit Weight γ (pcf)
Ground surface elevation: ~99 1/2										
Driller reported 10" of sandy topsoil FILL – dark brown – moist (SM-OL-FILL)										
Fine to medium sand FILL; some wood debris, trace gravel, silt and organics – dark brown – moist – loose (SP-MISC. FILL)			1	SS	4				47	
Fine to medium sand FILL; trace to some silt, few concrete, coal and wood pieces – brown – moist to wet – medium dense to very loose (SP-MISC. FILL)	5		2	SS	20				6	
			3	SS	2				79	
Fine to medium SAND; trace to some silt and organics – brownish gray to dark brown – wet – loose (SM-OL)			4	SS	5				20	
Boring terminated at 10'	10									

NOTE: Changes in soil stratification indicated by lines are approximate. In situ, the transition between materials maybe gradual unless otherwise noted. The bored hole was backfilled with natural soil.

WATER LEVEL OBSERVATIONS		BORING		SHEET
~9' While Sampling or Drilling		Rig: RD-CME-85	Foreman: J. RAU	1 of 1
DC @ 7.5' Immediately After Completion		Started: 1-12-01	Drawn By: R. MARR	
____ @____ After Completion		Completed: 1-12-01	Approved: U. GOLLAPUDI	



= APPROXIMATE SOIL BORING LOCATION

N
PLAN

SOIL BORING PLAN

1"=120'

		APPROXIMATE LOCATION	
NO.	DEPTH	POINT A	ELEVATION
B1	40'	460'N – 290'E	100.0
B2	40'	100'N – 320'E	100.0
B3	60'	230'N – 195'E	106.5
B4	85'	15'N – 115'E	101.5
B5	10'	90'S – 0'E	99.5

REV. #	DATE	DESCRIPTION	BY
	03-22-01	BORING LOCATIONS	MN

STATUS/REVISIONS

WAK Engineers-Architects-Consultants
William A. Kibbe & Associates, Inc.
1475 S. Washington Avenue, Saginaw, MI 48601

PROJECT:
MARINA PLACE CONDOMINIUMS
BAY CITY, MICHIGAN

CLIENT: MARINA PLACE, L.L.C.

DRAWN BY: BB
DESIGNED BY: MN
CHECKED BY: MN
APPROVED BY: MN
PROJECT NUMBER: 00-881-0626

SHEET TITLE: SOIL BORING PLAN
DRAWING NUMBER: SB
SHEET 1 OF 1