Chinook GeoServices Inc.

September 18, 2006

M.A.N. Developments
6107 Southwest Murray Boulevard, Suite 424
Beaverton, Oregon 97008
Attention: Mr. Mike Nelson

Subject: **Geotechnical Engineering Evaluation Report**
Proposed 92-Unit Townhome Development
Marine Drive, Wheeler, Tillamook County, Oregon
CGI Report No. 06-035-1

Dear Mr. Nelson:

Chinook GeoServices, Inc. (CGI) is pleased to submit our Geotechnical Engineering Evaluation Report for the proposed townhome development located in Wheeler, Oregon. This report includes the results of our literature research, field reconnaissance and subsurface exploration, laboratory testing, and geotechnical engineering conclusions and recommendations for the proposed construction, as well as recommendations for general site development.

We appreciate the opportunity to perform this study and look forward to continued participation during the design and construction phases of this project. Please contact Marcy Boyer or Warren Krager in our office at 360-695-8500 if you have any questions or if we may be of further service.

Respectfully submitted,

CHINOOK GEOSERVICES, INC.

Marcella M. Boyer, P.E., G.E.
Principal Geotechnical Engineer

R. Warren Krager, R.G., C.E.G.
Principal Engineering Geologist

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GEOTECHNICAL ENGINEERING EVALUATION REPORT

For the

Proposed 92-Unit Townhome Development
Marine Drive, Wheeler, Tillamook County, Oregon

Prepared for

M.A.N. Developments
6107 Southwest Murray Boulevard, Suite 424
Beaverton, Oregon 97008

Prepared by

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CGI Report No. 06-035-1

September 18, 2006

Chinook GeoServices, Inc.

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Marcella M. Boyer, P.E., G.E.
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1.0 EXECUTIVE SUMMARY

A subsurface exploration program and a geotechnical engineering evaluation of seismic liquefaction potential and subsurface conditions has been completed for the proposed 92-unit townhome development to be constructed at the north terminus of Marine Drive in Wheeler, Oregon. Nine test pits were excavated across the site to investigate the condition and thickness of the upper fill. To investigate the deeper native soils, two mud rotary borings were drilled—one each at the north and south ends of the site. Selected soil samples from both the borings and test pits were tested in the laboratory to determine index, compressibility, and strength properties of the soils encountered. In general, the subsurface conditions consisted of a thin layer of topsoil (no more than a foot in thickness), overlying 8½ to 17 feet of variable density, organic fill. The fill was underlain by soft/loose, compressible peat, sand, silt and clay to a depth of at least 101 ½ feet in boring B-1 and 33½ feet in boring B-2. We recommend groundwater be considered to be as shallow as 6 to 10 below existing grade, depending upon weather and tidal conditions.

The site has several geotechnical engineering concerns including:

- Soft, compressible fill 8½ to 17 feet in thickness that contains wood, logs and stumps and could settle several inches
- Soft, native peat and organic soils that are extremely compressible and could settle an additional 4 to 8 inches
- Soils that could liquefy and settle up to about 10 to 15 inches and slide laterally up to about 5 to 16 feet if the site is shaken by a major design earthquake
- The site is within the tsunami inundation zone and will be flooded if a major tsunami strikes the coastline

The proposed buildings cannot be supported on conventional shallow foundation systems because they cannot tolerate the excessive settlements caused by the geotechnical engineering concerns outlined above. We understand that the project team has already determined that pile foundation support is not economically viable. Based on these constraints, we recommend the use of either shallow, rigid, reinforced concrete raft or mat foundations supported on lightweight geofoam blocks. The raft or mat foundations will spread the building loads out uniformly across the building pads. The geofoam will replace existing site soils so that the weight of the new buildings will not be "felt" by the underlying soils. Ideally, this would mean that there would not be excessive settlement of the foundations. However, because of the complexity of the subsurface soil conditions, we recommend some excessive settlement (i.e. several inches) be planned for. As such, we recommend the use of flexible utility connections and we recommend the building entrances be constructed at least 1 to 2 feet high to allow for some building settlement. Site preparation of the building pads prior to geofoam block placement will require the removal of organics (wood debris) where more concentrated.

Details related to foundation design, and site preparation and construction considerations are included in subsequent sections of this report. The owner and/or designer should not rely solely on this Executive Summary and must read and evaluate the entire contents of this report prior to using our engineering recommendations to prepare the design/construction documents.

2.0 PROJECT INFORMATION

2.1 Project Authorization

Chinook GeoServices, Inc. (CGI) has completed a geotechnical engineering evaluation for the proposed townhome development to be constructed at the northern terminus of Marine Drive in Wheeler, Tillamook County, Oregon. Our services were authorized by Mr. Mike Nelson, owner of M.A.N. Developments on July 19, 2006 by signing CGI Proposal No. 06-P045 dated July 7, 2006. Our services were completed in general accordance with our proposal.

2.2 Project Description

Our current understanding of the project is based on limited information provided to us by Mr. Nelson including an undated, untitled site development plan. The 5-acre waterfront site will be developed with 17, three-story townhome buildings with a total of 92 living units. Four of the townhome buildings will also have some commercial space. There will also be 3 commercial buildings. The commercial space all appears to be located in the middle of the project. The building footprints will range in size from about 3,500 to 7,000 square feet. A pool—presumably below grade—is shown on the north-central portion of the site development plan. A promenade will be constructed at the west side of the middle of the site.

Below ground construction is not planned to our knowledge with the exception of utilities, and possibly the pool. According to Mr. Nelson, the site will not require any new fill to raise the grade elevation except at the promenade, where up to about 6 inches of fill will be needed. Finish floor of the proposed buildings roughly matches existing grade so no fill will be required under the buildings.

Building loads have not been provided to us. For the purposes of our engineering analyses, we have assumed the buildings will have maximum wall, column and floor loads of 4 kips per linear foot, 75 kips, and 0.150 kips per square foot, respectively. Due to the complexity of this project, it will be imperative that actual building loads be provided to us for review at some point during the design process.

2.3 Purpose and Scope of Services

The purpose of this study was to evaluate seismic liquefaction potential and geotechnical engineering subsurface conditions at the site to enable an evaluation of acceptable foundation recommendations for the proposed townhomes. We understand that you have already determined that supporting the proposed buildings on piles is not economically feasible and we have not included this alternative in our scope of services. Our scope of services included advancing 2 borings using mud rotary drilling techniques to investigate the native soil conditions, 9 test pits to investigate the overlying fill conditions, select laboratory testing, engineering analysis and preparation of this report.

3.0 SITE AND SUBSURFACE CONDITIONS

3.1 Site Description

A street address for the property was not available to us. The site is roughly located at the north terminus of Marine Drive and the west terminus of Hemlock Street. It is bordered by Port of Tillamook Bay Railroad tracks and Highway 101 to the east, Nehalem Bay to the west and north, and a residence and the Wheeler Marina (278 Marine Drive) to the south. The site is relatively level. Several feet of fill has been placed across the site in the past. Apparently, one of the former uses at the site was to transport and store logs. A review of an August 22, 2000 aerial map on the www.terraserver-usa.com website shows that logs were not being stored at that time. The site is also clear of any structures at that time. A July 1, 1985 topographic map from the same website indicates that several structures were located on the property. During our site investigation, we observed buried concrete foundations and concrete slab remnants at the surface of the site.

3.2 Geologic Background

According to the 1994 "Geologic Map of the Tillamook Highlands, Northwest Oregon Coast Range," (Open File Report 94-21) prepared by the US Department of the Interior, US Geologic Survey (USGS), the site geology consists of fluvial and estuarine Holocene deposits (deposits within the last 11,000 years). These deposits typically consist of unconsolidated clay, silt and gravel alluvium deposited along rivers and streams and stabilized tidal flat mud, sand and peat in Nehalem and Tillamook Bays. Discounting the upper fill, the site geology encountered in our borings and test pits appears to be consistent with the geologic conditions described above.

3.3 Geotechnical Subsurface Investigation

CGI completed 2 mud rotary borings and 9 backhoe test pits to evaluate the subsurface conditions (reference Appendix C for a description of the field investigation procedures). Based on the borings and test pits, the site subsurface consisted of the following soil units:

TOPSOIL – Brown, slightly moist silt topsoil was encountered in all exploration locations except boring B-2, which was located in a gravel parking area. The topsoil unit was generally firm and ranged in thickness from about 2 to 12 inches.

FILL – Variable fill was encountered immediately below the topsoil layer in all borings and test pits. The fill consisted of varying amounts of clay, silt, sand, gravel and cobbles. Varying degrees of construction debris, organics and wood pieces were also encountered in all test pits (see Photos 4 and 5 in the Photo Log attached for examples of wood pieces encountered). The fill thickness ranged from 8½ to 17 feet. It appears the fill was imported from a number of different sites based on the high variability of the material. N-values and pocket penetrometer readings indicate the fill was not fully compacted, although the general trend was that the upper portion of fill was much firmer and the bottom of the fill was noticeably softer/looser. The fill ranged from very soft to very dense. The moisture condition of the fill ranged from slightly moist near the ground surface to wet near the bottom of the fill. Note that TP-6 did not extend through this fill stratum into the native soil due to practical digging refusal on very dense gravel fill. In TP-8 it appears that we encountered the remnants of a buried building foundation at a depth of about 6 feet (see Photos 7 and 8 attached).

PEAT – Test pits TP-4 and TP-5 each encountered a 2-foot thick layer of fibrous peat at a depth of 12 and 11 feet, respectively. The peat was free of soil and appeared to be buried marsh grass, similar to what is observed west and north of the site. Peat was also observed in the native silt and clay soils immediately beneath the fill placed across the site, however, the peat content appeared to be small—10.6 and 17.7 percent in the samples tested (see Photo 6 attached).

SILT AND CLAY WITH ORGANICS – All borings and test pits (excluding TP-6 because it did not penetrate through the fill) encountered a soil unit that was predominantly silt and clay immediately beneath the peat layer in TP-4 and TP-5 and beneath the fill stratum of all other test pits and borings. This soil unit consisted of both low and high plasticity clay and silt based on Atterberg limits testing. The top couple of feet of this stratum generally included some wood pieces and moderate peat content. The organic content decreased with depth. It is likely the wood pieces were pushed into the native stratum by the weight of the fill above. In boring B-2, this silt/clay stratum included gravel and trace sand below a depth of 25 feet. Based on Standard Penetration Tests and pocket penetrometer readings, the strength of this unit was generally very soft to medium stiff, except in boring B-2 at the south end of the site where it was very stiff to hard starting at a depth of 25 feet. Moisture content of the samples tested ranged from 28 percent to 79 percent.

SAND WITH SILT AND TRACE ORGANICS – Boring B-1 was extended significantly deeper than the other boring and test pits. From a depth of 45 feet to 75 feet below grade, sand with silt and trace organics (shells and wood) was encountered. This stratum was generally wet and very loose to loose.

SILT WITH TRACE ORGANICS – From 75 feet to the termination depth of 101½ feet below grade, silt with trace organics was encountered in boring B-1. The stratum was generally very soft to medium stiff in consistency with moisture contents ranging from 47 percent to 57 percent.

The above subsurface description is of a generalized nature to highlight the major subsurface stratification features and material characteristics. The boring and test pit logs included in the appendix should be reviewed for specific information at individual exploration locations. These records include soil descriptions, stratifications, penetration resistances (automatic hammer equivalent), locations of the samples, and laboratory test data. The stratifications shown on the exploration logs represent the conditions only at the actual boring and test pit locations. Variations may occur and should be expected between exploration locations. The stratifications represent the approximate boundary between subsurface materials and the actual transition may be gradual. The samples that were not altered by laboratory testing will be retained for 60 days from the date of this report and then will be discarded.

3.4 Groundwater Information

Groundwater levels are extremely difficult to accurately measure in mud rotary borings. We were, however, able to observe groundwater levels in the test pits at the time of excavation. Seepage was generally observed in the test pit sidewalls at a depth of 6 feet to 11 feet below existing grade. Standing groundwater was generally observed in the test pits at depths of 8 feet to 16 feet. Because the site soils within the seepage zone were generally cohesive with low permeability, it is probable that if the test pits were allowed to stand open long enough, the standing water would have risen close to the seepage depth. For planning purposes, we recommend the groundwater depth be anticipated to be about 6 to 10 feet below existing grade. Some fluctuations of groundwater levels should be anticipated with changing climatic conditions, tides, and/or changes in surface topography, construction activities and site use.

4.0 GEOTECHNICAL EARTHQUAKE ENGINEERING EVALUATION

4.1 Seismicity

There are three major types of earthquakes that can occur in Oregon: deep interplate or subduction zone earthquakes, moderately deep intraplate earthquakes and shallow crustal earthquakes. A review of the 1994 USGS map for this area (referenced earlier in this report) indicates the presence of several faults in the area. However, current understanding of the activity of these faults and the potential for an earthquake to occur within the life of the proposed structure is not very well understood. Additionally, there may be yet undiscovered faults in the area capable of generating significant ground motion at the subject site.

The Cascadia Subduction Zone, located approximately 100 kilometers (km), or roughly 60 miles, off the Oregon and Washington coasts, is an immense thrust fault and a potential source of earthquakes large enough to cause significant ground shaking at the subject site and potentially throughout western Oregon and Washington. Research over the last several years has shown that this offshore thrust fault zone has repeatedly produced large earthquakes every 300 to 700 years. Geologic research of ancient Japanese tsunami records along with dendrochronology (tree ring dating techniques) has established that the last large Cascadia Subduction Zone earthquake occurred in January of 1700 AD. Given the date of the last earthquake in 1700 and the average return interval, it is generally considered that Oregon could be impacted by a very large subduction zone earthquake at any time in the next 400 years or so. Although researchers do not agree on the likely magnitude of the next Cascadia Subduction Zone thrust fault earthquake, it is widely believed that an earthquake of moment magnitude (Mw) 8.5 to 9.5 is possible. The duration of strong ground shaking is estimated to be up to about two or three minutes, with minor shaking lasting several minutes. Aftershock earthquakes could continue for weeks or months after the main thrust fault rupture. It is also possible that large areas of the coastline will experience several feet of rapid, permanent subsidence as a result of a large Cascadia Subduction Zone earthquake. Remnants of salt water flooded and killed forests and rooted stumps in the surf are still present after the last Cascadia Subduction Zone earthquake. These geologic features provide evidence that up to 6 or more feet of coseismic subsidence may be possible. This amount of subsidence coupled with sea level rise from global warming will render many currently low lying coastal areas uninhabitable without substantial engineering solutions.

Additional earthquake sources in this region include fault ruptures within the subducting oceanic plates (interplate earthquake) and within the overlying North American continental crustal plate (crustal earthquake). Intraplate earthquakes originate at depths on the order of about 50 km (30 miles) within the remains of the Juan de Fuca Plate. These large earthquakes have occurred with historical frequency in western Washington and to a lesser extent in western Oregon. These earthquakes range up moment magnitude (Mw) 7.5 and have caused widespread damage in the southern Puget Sound and northwest Oregon region in 1949, 1965, 1971, and 2001.

Crustal earthquakes are relatively shallow, occurring within 10 to 20 km (6 to 12 miles) of the ground surface within the North American Plate. Oregon has experienced at least two significant crustal earthquakes in the recent past—the Scotts Mills (Mt. Angel) earthquake (Mw 5.6) on March 25, 1993 and the Klamath Falls earthquake (Mw 5.9) on September 20, 1993.

4.2 Seismic Hazards

While we have not performed a detailed site-specific seismic evaluation, we have done a preliminary analysis of seismic hazards which could potentially impact the site. These seismic hazards include:

- Severe ground shaking
- Ground surface subsidence
- Fault rupture
- Liquefaction, dynamic settlement and lateral spread
- Seiche
- Tsunami
- Earthquake-induced landslides

Based on the historical seismic activity in Oregon, it is possible that the proposed development will be impacted by *severe ground shaking* within its useful life. The severity of ground shaking at the site will be determined in part by the source of the earthquake (i.e. Cascadia Subduction Zone versus local shallow crustal). Structural design parameters required to address ground shaking are provided in Section 4.4 below.

The risk of *ground surface subsidence,* as previously discussed in Section 4.1, is possible due to either a Cascadia Subduction Zone earthquake, which causes the entire coastline to drop several feet, or liquefaction.

Because of the lack of historical data on fault activity in the area, *fault rupture* is difficult to predict. There was no evidence of past fault rupture at this site during our subsurface investigation, although the conditions at the site would make it difficult to observe fault rupture.

Based on our subsurface investigation and laboratory testing, *liquefaction* is possible at this site. Our liquefaction analysis will be discussed in detail in the next section.

A *seiche* is a standing wave in an enclosed or partially enclosed body of water. Ground shaking during an earthquake can cause the body of water to slosh back and forth, repeatedly flooding the margins of the body of water. Seiches can occur on lakes, reservoirs, bays and seas. Considering the close proximity of this site to Nehalem Bay, this site is at risk of being flooded by a seiche.

A *tsunami*, or seismic sea wave, is produced when a fault under the ocean floor shifts vertically, displacing the seawater above it. The Tsunami Inundation Map of the Nehalem Quadrangle, (DOGAMI Open File Report O-95-17), included as Figure 4, indicates that the site is within the tsunami inundation zone. Wave run-up heights are anticipated to range from 15 feet to 31 feet above sea level (Priest, 1995). This model is based on an estimated subduction zone earthquake of Mw=8.8 to 8.9 and does not include storm wave influence. **This site should be considered at significant risk to tsunami inundation.**

4.3 Liquefaction, Dynamic Settlement and Lateral Spread Evaluation

Liquefaction occurs when saturated deposits of loose to medium dense, cohesionless, fine-grained soils—generally sands and low plasticity silts and clays—are subjected to strong earthquake shaking. If these deposits are saturated and cannot drain rapidly, there will be an increase in pore water pressure. With increasing oscillation, the pore water pressure can increase to the value of the overburden pressure. The shear strength of a cohesionless soil is directly proportional to the effective stress, which is equal to the difference between the overburden pressure and the pore water pressure. Therefore, when the pore water pressure increases to the value of the overburden pressure, the shear strength of the soil reduces to zero, and the soil deposit liquefies (i.e. acts like a fluid).

We performed our liquefaction analysis using the largely empirical *Simplified Procedure* originally developed by Seed and Idriss (1982), and since updated several times, including major revisions in 2004 (Cetin et al.). Our soil strength parameters in our analyses were based on the SPT and pocket penetrometer data collected in our borings.

For our liquefaction analysis, we assumed a groundwater depth of 7.5 feet below the existing ground surface. Based on our soil explorations, laboratory testing, and analysis, liquefiable soil was present in our borings below a depth of about 15 feet. We limited the depth of liquefiable soils to 50 feet below the ground surface, which is generally the standard of practice. Therefore, our liquefiable layer was from 15 feet to 50 feet.

Our evaluation was modeled after a large Cascadia Subduction earthquake with a moment magnitude of 8.5. While a greater magnitude earthquake could occur, the *Simplified Procedure* limits the magnitude to 8.5. Based on our engineering analyses and data collected in boring B-1 at the north end of the site, we estimate dynamic settlement resulting from liquefaction could be as much as 10 inches to 15 inches.

Boring B-2 at the south end of the site indicated the presence of high plasticity clay soils below a depth of about 15 feet. High plasticity clay soils are not considered liquefiable. A 7 foot to 8 foot thick, loose layer of course sand above the clay layer is considered liquefiable. Our analysis indicates up to about 3 inches or 4 inches of dynamic settlement where conditions are similar to B-2. Differential dynamic settlement is generally estimated at one-half to two-thirds of total dynamic settlement.

We performed lateral spread analyses in accordance with the method by Youd et al. (1999). Based on those results, we estimate as much as 5 feet of lateral spread could occur where subsurface soil conditions are similar to boring B-2 at the south end of the site and up to 16 feet of lateral spread could occur where subsurface conditions are similar to boring B-1 at the north end of the site. Our estimations are based on a very large (M_w 9) Cascadia Subduction Zone earthquake. The amount of lateral spread occurring at the site would be different if the earthquake magnitude or source are different.

4.4 2004 SOSSC Seismic Design Parameters

In accordance with Table 1615.1.1 of the 2004 State of Oregon Structural Specialty Code (SOSSC), which is an amendment to the 2003 International Building Code (IBC), we recommend a **Site Class F** for this site when considering the average of the upper 100 feet. We recommend Site Class F because the site soils are potentially liquefiable. According to the 2002 USGS Earthquake Hazards website http://eqint.cr.usgs.gov/eq-men/html/lookup-2002-interp-06.html, the Peak Ground Acceleration (PGA) is 0.56, and the maximum considered earthquake (MCE) ground motions for the site (45.69374 degrees latitude and -123.88106 degrees longitude) are **S_S=1.324g** and **S_1=0.675g** (for Site Class B and 5 percent critical damping). The USGS website values are a more accurate interpolation of the values presented in Figure 1615(1) of the IBC.

In accordance with Note b in Tables 1615.1.2(1) and 1615.1.2.(2), Site Coefficients F_a and F_v for liquefiable soils may be obtained from the 2 referenced tables based on the Site Class determined without regard to liquefaction provided the building period is not greater than 0.5 seconds. We anticipate the three-story buildings being considered will have a building period not greater than 0.5 seconds. The building period can be estimated by multiplying the number of building stories times 0.1 seconds, or about 0.3 second period for the proposed three-story buildings. Should the project structural engineer determine that any of the building periods are greater than 0.5 seconds, then a site-specific geotechnical investigation and dynamic site response analysis would be required by the code.

For this site, ignoring liquefaction as justified above, we can use the Site Class E coefficients. Therefore, Site Coefficients are **F_a=0.9** and **F_v=2.40**. The adjusted MCE ground motions are **S_{MS}=1.192g** and **S_{M1}=1.620g** (for Site Class F and without regard to liquefaction). The return interval for these ground motions is 2 percent probability of exceedance in 50 years.

A site-specific seismic hazard study was beyond the present scope of services for this project.

5.0 GEOTECHNICAL ENGINEERING EVALUATION

5.1 Geotechnical Engineering Discussion

Based on our evaluation, the geotechnical engineering soil conditions are poor for this site, however the proposed development is suitable, provided the recommendations in this report are incorporated into the design and construction of the project and the risks associated with site development are understood. We understand from Mr. Nelson that supporting the proposed buildings on piles is not an economical option and that alternative foundation methods are necessary. The primary geotechnical factors influencing the proposed construction include:

1. **Variable Density Fill with Organics and Debris**. The native ground at this site likely use to be at the elevation of the swamplands immediately north of the site. At some point, it appears the site was filled to its current elevation, possibly in stages based on the different fill materials used. The fill was generally about 8½ feet to 17 feet in our borings and test pits and, for the most part, does not appear to have been properly compacted. Much of the organic content in the fill was trace to minor and is not anticipated to negatively impact the proposed construction. However, in some test pits, a portion of the fill consisted entirely of organics (wood pieces). It appears these might be buried wood piles. This occurred at TP-1 (4 to 7 feet below grade), TP-2 (3 ½ to about 8 feet below grade), TP-6 (5 to 8 feet below grade), and TP-7 (7 to 10 feet below grade) and could be present elsewhere throughout the site.

   The presence of a relatively thick covering of poorly compacted fill and organics will cause the proposed buildings to settle excessively when considering both total and differential settlement. Settlement due to organics could continue slowly for years as the material decomposes. The amount of total and differential static settlement is difficult to predict because the fill is so variable, but total static settlement could be several inches, with maximum differential static settlement as much as 90 percent to 100 percent of total settlement.

2. **Soft, Compressible Peat and Organic Clay**. All of the borings and test pits encountered organic soil. The upper fill primarily contained wood pieces, logs and stumps, while the underlying native soil primarily contained peat and rootlets. The organics in the upper fill will be prone to static settlement from building loads as well as long-term static settlement due to decay. The peat will not likely decay because it appears to be below the water table, however it is extremely compressible. Analyses of one-dimensional consolidation lab tests indicate the potential for up to about 4 to 8 inches of primary total static settlement that could take several years to occur.

3. **Potentially Liquefiable Soil**. In Section 4.3 above, we estimated that potentially liquefiable soil was present in boring B-1 at the north end of the site from about 15 feet to 50 feet. The liquefiable soil layer in B-2 was considered from 7½ to 15 feet. The test pits did not extend deep enough to evaluate liquefaction. Based on our evaluation of boring B-1, we estimated

up to 10 inches to 15 inches of potential total dynamic settlement. We estimated up to 3 inches to 4 inches of dynamic settlement in boring B-2. Differential dynamic settlement is generally estimated at ½ to 2/3 of total dynamic settlement. These settlement magnitudes would be extremely destructive to buildings supported on conventional shallow foundations.

4. **Lateral Spread**. Based on our analyses, we estimated up to 5 to 16 feet of lateral spread should a major earthquake impact the site.

5. **Seiche or Tsunami Event**. Damage to the property from a seiche or tsunami could be extensive.

6. **Potential Methane Gas Generation from Buried Organic Decomposition**. While methane gas risk and mitigation is not part of our expertise as geotechnical engineers, we should caution that buried organics like those found on this site are known to generate harmful methane gas, which then works its way to the surface and can collect in crawl spaces and basements. Since we do not anticipate that the proposed construction will include crawl spaces or basements, methane gas hazard does not appear to be a problem. But we do recommend you consult someone with expertise in mitigating the risk of methane gas as a precaution.

Based on the above geotechnical factors, and the understanding that a pile foundation system is not an acceptable alternative, we recommend the proposed buildings be supported on either shallow raft or mat foundations. A conventional shallow foundation system is not appropriate for this site.

A raft foundation normally consists of a reinforced concrete slab which extends over the entire loaded area. It is then stiffened with a matrix of criss-crossing reinforced concrete ribs or beams. Where the raft may be variable thickness, a mat foundation is generally uniform thickness throughout with 2 layers of reinforcing steel. The raft foundation system typically uses less concrete than a mat and may be more economical. However, excavation for the raft system can be more complicated. Ultimately, the foundation system selected needs be rigid enough to compensate for several inches of differential static and dynamic settlement.

With a raft or mat foundation, the buildings will still settle several inches over time. This means that eventually exterior flatwork (i.e. sidewalks, patios, driveways) may not match up well with building entrances. We recommend that exterior flatwork not be structurally connected to the buildings. We also recommend consideration be given to raising the buildings 1 to 2 feet above exterior grade and entering the buildings with steps. Then when the buildings settle over their useful life, the steps into the buildings can easily be modified as necessary. We also recommend that the buildings be constructed as early as possible in the construction schedule and then wait as late as possible in the construction schedule to construct exterior flatwork and attach underground utilities at the perimeter of the buildings. Specially designed, flexible utility connections should also be used to compensate for future building settlement.

We have considered methods for reducing the amount of potential static settlement. These methods include surcharging the site prior to construction and load compensation. The surcharge program would consist of temporarily placing fill on each building pad that is greater than the weight of the proposed building. We recommend a surcharge of at least twice the weight of the building. Based on our consolidation lab tests, it could take a couple of years for a surcharge program to be completed. While the surcharge will mitigate short-term settlement of the soft, compressible soil, it will not mitigate static settlement that will occur over several years due to decomposition of the upper organic soils and the slow settling peat and organic soil. So surcharging will significantly reduce the total amount of static settlement but it will not eliminate it completely. More discussion on the surcharge alternative is presented in Section 5.5. However, given the fact that it could take a couple of years for most of the static settlement to occur, we anticipate the surcharge program may not meet the project schedule needs.

The load compensation program would consist of "unloading" each building pad by at least the total weight of the building. This would normally be done by lowering the building pad grades, but we do not anticipate that this is feasible at this site. Therefore, we recommend consideration be given to replacing site soil beneath the building pads with lightweight rigid cellular polystyrene geofoam (geofoam). This material comes in block shapes and typically has a density of 1 to 2 pounds per cubic foot (pcf), compared to the site soils to be replaced, which have a density on the order of 110 to 120 pcf. Depending on the weight of the proposed buildings, the site may need to be excavated several feet and replaced with geofoam blocks. Ideally, this would eliminate all settlement concerns, except the potential settlement due to liquefaction. However, due to the complexity of the subsurface conditions, we recommend the design team assume that the buildings will still experience total static settlement of several inches. As such, we recommend the use of flexible utility connections and consideration be given to constructing the building entrances 1 to 2 feet higher than the adjacent grade so there is room to settle without interrupting the entrances. More detailed design discussion on this alternative is presented in Section 5.6.

In summary, we anticipate that the buildings will be supported on rigid, reinforced concrete raft or mat foundations bearing on geofoam blocks after the major concentrations of organics (wood, logs, stumps) have been removed from the upper fill. Ideally, this system will mitigate excessive settlement except dynamic settlement caused by an earthquake.

5.2 Site Preparation

We recommend that topsoil, vegetation, roots, wood, stumps and old buried foundations and utilities in the construction areas be stripped from the site. A representative of the Geotechnical Engineer should evaluate the near surface soils and determine of additional stripping or removal of previous improvements is needed at the time of construction. Utilities should be located and rerouted as necessary and any abandoned pipes or utility conduits should be removed to inhibit the potential for subsurface erosion. Utility trench excavations should be backfilled with properly compacted structural fill which is constructed as outlined in Section 5.3 of this report.

We recommend woody debris material be removed where the wood content is significant (i.e. TP-1, TP-2, TP-6 and TP-7). However, because the buildings will be supported on a rigid foundation that will mitigate settlement due to decomposition of the wood over time, it is not required that all organics be removed from the existing fill. Organic removal should be addressed by a representative of the Geotechnical Engineer during earthwork construction.

After stripping and excavating to the proposed subgrade level, building areas to support the building or pavement should be proofrolled with a heavily loaded tandem axle dump truck or similar rubber-tired vehicle. Soils that are observed to rut or deflect excessively under the moving load, or are otherwise judged to be unsuitable should be excavated and replaced with properly compacted fill. The proofrolling activities should be witnessed by a representative of the Geotechnical Engineer.

5.3 Fill Requirements

After subgrade preparation and observation have been completed, fill placement may begin if needed. The first layer of fill material should be placed in a relatively uniform horizontal lift on the prepared subgrade. Fill materials should be free of organic or other deleterious materials, have a maximum particle size less than 3 inches, be relatively well graded, and have a liquid limit less than 45 and plasticity index less than 25. The on-site, non-organic soils are suitable for use as structural fill. Structural fill should be compacted to at least 95 percent of modified Proctor maximum dry density as determined by ASTM D1557.

Fill should be placed in maximum lifts of 8 inches of loose material and should be compacted within the range of 3 percentage points below to 2 percentage points above the optimum moisture content value. If water must be added, it should be uniformly applied and thoroughly mixed into the soil by disking or scarifying. Each lift of compacted engineered fill should be tested by a representative of the Geotechnical Engineer prior to placement of subsequent lifts. The fill should extend horizontally outward beyond the exterior perimeter of the building and footings a distance equal to the height of the fill or 5 feet; which ever is greater, prior to sloping. Also, fill should extend horizontally outward from the exterior perimeter of the pavement a distance equal to the height of the fill or 3 feet; whichever is greater, prior to sloping.

5.4 Foundation Recommendations – Raft or Mat

As discussed in Section 5.1 above, we recommend the buildings be supported on rigid, shallow foundations that can tolerate several inches of differential settlement. This could consist of shallow, rigid raft or mat foundations. A conventional shallow foundation system should not be used for this site.

Structural design of reinforced concrete raft or mat foundations may include an elastic analysis of the foundation and bearing soil. This analysis uses a coefficient of vertical subgrade reaction (K_{v1}) to represent the elastic properties of the bearing soil. The coefficient of subgrade reaction is a function of soil type, compressibility and foundation size. K_{v1} represents the coefficient of subgrade reaction for a vertical load on a 1-square foot test plate. For the generally soft, compressible soils at this site, we recommend a soil coefficient of subgrade reaction (K_{v1}) of 43 kips per cubic foot (kcf)—roughly 25 pci—for the anticipated foundation bearing soils. The footings should be designed using a coefficient of subgrade reaction (K_v), which is determined for foundations supported on clayey soils, by dividing the subgrade modulus K_{v1} by the footing width (B)—see equation below. For a raft or mat foundation, we recommend setting B equal to the lesser of the smallest column spacing or foundation width.

$$K_v = K_{v1} / B$$

where: K_v = coefficient of subgrade reaction to be used by structural engineer, kcf
K_{v1} = coefficient of subgrade reaction for 1-foot wide footing, kcf
B = footing width (lesser of the smallest column spacing or foundation width), feet

For example, if K_{v1} is 43 pci as we recommended for this site, and the least column spacing is 20 feet, then the foundation coefficient of subgrade reaction to use in the structural design (K_v) is 2.15 kcf. K_v may be increased by a factor of 1.5 for transient seismic loading conditions.

Where the structural engineer designs rigid foundations using spring constants, each spring constant should be determined by calculating K_v as shown above and then multiplying K_v by the tributary area supported by the spring (spring constant units: pounds/foot).

Shallow foundations may be designed to resist lateral loads using a passive earth pressure based on an equivalent fluid density of 350 pounds per cubic foot on footings poured "neat" against firm in-situ soils or properly backfilled structural fill. The recommended passive earth pressure is an ultimate value and does not include a factor of safety. The structural engineer should apply a factor of safety of 1.5 to obtain the "allowable" value. The upper 1 foot of soil should be neglected when determining passive earth pressure resistance acting against the vertical face of footings, unless the adjacent ground is paved.

We do not recommend using frictional resistance between the base of footings and subgrade to resist lateral loads on foundations because we anticipate that over time, portions of the foundations will not be in contact with the subgrade as the site settles. Because foundation sliding resistance cannot be used, it may be necessary to construct keyways in the foundations to increase the passive resistance.

Foundations exposed to weather should be embedded at least 12 inches below the adjacent exterior grade to prevent frost heave. Provided the bottom of the foundation conservatively terminates no deeper than 7 ½ feet below existing grade, we do not anticipate that the foundation will need to be designed to resist buoyant forces due to groundwater.

5.5 Surcharge

The surcharge program, if utilized, would consist of temporarily placing fill on each building pad that is at least twice the weight of the proposed building. To speed up the process, additional fill height can be added. Once total building loads are provided by the project Structural Engineer, we can provide final recommendations for the surcharge volume required. For preliminary planning purposes, assuming surcharge fill material with an in-place unit weight of about 100 pcf and three-story buildings with a total load of about 100 psf per floor, the minimum surcharge height would be about 6 feet.

The following are general specifications for surcharge construction:

- Topsoil beneath the surcharge should be removed and wasted or stockpiled for use in landscape areas on-site.
- The site should be filled to or above the desired site grade with structural fill as described in the Site Preparation section of this report.
- A 2-foot thick layer of clean, free-draining material such as sand should be placed over the surcharged pads to facilitate drainage. The surcharge process will likely generate a lot of water at the base of the surcharge fill as the underlying soft/loose soils are compressed.
- Settlement plates should be placed across the site area to be preloaded, as indicated by the geotechnical engineer, to allow monitoring of the consolidation.
- The material used for the surcharge should generally be granular (i.e. silty sand, sand or gravel) so that it is easier to place.
- The surcharge fill should be moderately compacted with a large vibrating smooth drum roller but it does not need to be compacted to meet a structural fill requirement (i.e. at least 95 percent) since it will removed.
- The surcharge fill should be placed and the surface sloped to provide runoff of precipitation. Side slopes should be no steeper than 1 horizontal to 1 vertical (1H:1V).
- The surcharge should extend out laterally at least 5 feet beyond the building footprints.
- Monitoring of the surcharge with survey readings should take place on a regular basis. This data should be forwarded to the Geotechnical Engineer for evaluation so that it can be determined when the surcharge program is complete (i.e. the surcharge load has essentially stopped settling). Based on our consolidation lab tests, it could take a couple of years for a surcharge program to be completed.

5.6 Geofoam

The geofoam material should comply with ASTM D6817-02, "Standard Specifications for Rigid Cellular Polystyrene Geofoam." Further information is needed about the actual total building weights and footprint before we can provide our final geofoam material and thickness recommendations, but we anticipate that either an EPS15, EPS22, or EPS29 material would be appropriate. Each of these products has a different unit weight and compression resistance (reference Table 1 in ASTM D6817 for minimum unit weight and compressibility characteristics for different contact pressures). Selection of the final product should be based on optimizing the geofoam unit weight and compressibility. Provided the EPS15 material meets the settlement criteria for the given contact pressure, it would be the most cost effective. EPS15 has a unit weight of about 1 pound per cubic foot, and will compress no more than 1 percent when loaded up to about 500 psf.

Geofoam is adversely affected by exposure to ultra-violet light. As such, it should be embedded in the ground and not exposed to sunlight. Prior to installation, we recommend the geofoam supplier perform compression testing on their product to confirm its compressibility. Previous experience indicates that the local geofoam suppliers use different quality polystyrene resin beads. The bead quality affects the geofoam compression resistance. The final recommendations for which geofoam product to use and how thick it should be should be based on the suppliers' compression testing.

For preliminary budgeting purposes, the EPS15 material is currently available from local suppliers for about $40 per cubic yard. For a building pad of about 7,000 square feet and a uniform foundation pressure of 300 pounds per square foot, roughly 720 cubic yards of geofoam would be required, at a cost of about $29,000.

Provided the bottom of the geofoam terminates no deeper than 7½ feet below existing grade, we do not anticipate that the foundation system will need to be designed to resist buoyant forces due to groundwater. We recommend a properly compacted granular leveling pad of at least 4 inches be placed between the native soil and geofoam.

6.0 CONSTRUCTION CONSIDERATIONS

CGI should be retained to provide observation and testing of construction activities involved in the foundation and earthwork related activities of this project. GCI cannot accept any responsibility for any conditions that deviate from those described in this report, nor for the performance of the foundation if not engaged to also provide construction observation and testing for this project.

6.1 Drainage and Groundwater Considerations

Water should not be allowed to collect in the foundation excavations or on prepared subgrades for structures during construction. Positive site drainage should be maintained throughout construction activities. Excavated areas should be sloped toward one corner to allow water removal.

The site grading plan should be developed to provide rapid drainage of surface water away from the building and to inhibit infiltration of surface water around the perimeter of the building and beneath the floor slabs. Careful consideration should be given to the potential impact of landscaped areas and/or sprinkler systems on adjacent foundations and floor slabs. Roof run off should be conveyed at least 10 feet away from the building prior to discharge upon unpaved surfaces.

6.2 Excavations

Temporary earth slopes and trenches should be cut in accordance with Department of Labor Occupational Safety and Health Administration (OSHA) guidelines. Job site safety is the responsibility of the project contractor.

In Federal Register, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, part 1926, Subpart P". This document and subsequent updates were issued to better insure the safety of workmen entering trenches or excavations. It is mandated by this federal regulation that excavations, whether they be utility trenches, basement excavations or footing excavations, be constructed in accordance with the new OSHA guidelines. It is our understanding that these regulations are being strictly enforced and if they are not closely followed, the owner and the contractor could be liable for substantial penalties.

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor's "responsible person", as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations.

We are providing this information solely as a service to our client. CGI does not assume responsibility for construction site safety or the contractor's compliance with local, state, and federal safety or other regulations.

6.3 Construction Dewatering

Ground water was estimated to be approximately 6 to 16 feet below the ground surface during our explorations. Groundwater should be maintained at least 2 feet below all excavations. We anticipate that pumping from sumps outside the limits of the excavation should control seepage and surface water ponding. Dewatering wells may be needed if the excavations were to remain open for an extended period of time.

6.4 Permanent Cut and Fill Slopes

Permanent cut and fill slopes should not exceed 2H:1V. We recommend that slopes that will be mowed not exceed 3H:1V. Structures should be set back a minimum of 10 feet from any major slope crest. Slope erosion should be prevented by being planted with sufficient vegetation or covered with other measures to prevent erosion which could reduce the minimum set back. We assumed that cuts and fills would be less than 2 feet for this project. We should be contacted to provide further recommendations if thicker cuts and/or fills are designed.

7.0 REPORT LIMITATIONS

The recommendations submitted in this report are based on the available subsurface information obtained by CGI and design details furnished by Mr. Nelson for the proposed townhome development. If there are any revisions to the plans for this project, or if deviations from the subsurface conditions noted in this report are encountered during construction, CGI should be notified immediately to determine if changes in the foundation recommendations are required. If CGI is not retained to review these changes, CGI will not be responsible for the impact of those conditions on the project.

The Geotechnical Engineer warrants that the findings, recommendations, specifications, or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed.

After the plans and specifications are more complete, the geotechnical engineer should be retained and provided the opportunity to review the final design plans and specifications to check that our engineering recommendations have been properly incorporated into the design documents. At this time, it may be necessary to submit supplementary recommendations. This report has been prepared for the exclusive use of M.A.N. Developments for the specific application to the proposed construction.